For Reference

NOT TO BE TAKEN FROM THIS ROOM

For Reference

NOT TO BE TAKEN FROM THIS ROOM

Ex Libris
Universitatis
Albertaensis

UNIVERSITY OF ALBERTA
QUAECUMQUE VERA

Digitized by the Internet Archive
in 2018 with funding from
University of Alberta Libraries

https://archive.org/details/Ottley1962

THE UNIVERSITY OF ALBERTA

SWELLING PROPERTIES OF EXPANSIVE CLAYS

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE

DEGREE OF MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

by

HORACE E. R. OTTLEY

EDMONTON, ALBERTA

MAY, 1962

ABSTRACT

Conventional methods of stability analysis have been found inadequate for predicting the stability of slopes in over-consolidated clay, and the proposal has been made, that the reliability of these methods may be improved by including the swelling pressure of the soil in the definition of effective stress. The purpose of this investigation is to try and determine the value of the proposal.

A review is given of studies that have been made on the electro-chemical bonding forces acting in soils, of the mechanism of swelling and of current methods used to measure swelling pressure.

A description of the apparatus is given, and the test procedures are outlined. The test results are discussed, and a comparison is made between the strength envelopes obtained in the Standard tests and in the Constant Volume tests. No conclusive evidence has been found to prove or disprove the proposal.

Recommendations have been made for further research.

ACKNOWLEDGEMENTS

This thesis has been prepared under the direction of Professor S. R. Sinclair, to whom the author is indebted for the guidance and criticisms given throughout its preparation.

The author wishes to acknowledge the financial support granted by the Joint Highway Research Programme which is sponsored by the Department of Highways of the Province of Alberta, the Research Council of Alberta Highways Division, and the Department of Civil Engineering of the University of Alberta.

H. E. R. O.

EDMONTON

May, 1962

TABLE OF CONTENTS

PAGE

ABSTRACT i

ACKNOWLEDGEMENTS ii

LIST OF TABLES vi

LIST OF PLATES vii

LIST OF FIGURES viii

GLOSSARY x

CHAPTER I. THE PROBLEM AND SCOPE OF THE RESEARCH 1

Introduction 1
Summary of Paper by Hardy, Brooker and Curtis, 1960 3
Purpose and Scope of the Research 11

CHAPTER II. THE STRUCTURE AND BEHAVIOUR OF CLAY SOILS 13

The Clay Minerals 13
The Structure of Clay Soils 14
The Behaviour of Clay Soils
a. The influence of adsorbed cations 16
b. The nature and importance of water 17

CHAPTER III. MECHANISMS OF SWELLING 20

Definition and Early Concepts 20
Theories Based on the Physico-Chemical properties of the soil.
a. The Osmotic Pressure theory 21
b. The Auto-Disintegration theory 23

PAGE

Theories based on the Physical Properties of the Soil
a. The Soil Water Potential theory....24
b. The Mechanical theory....25
c. The Locked-Pressure theory....27

CHAPTER IV. THE SWELLING PRESSURE OF EXPANSIVE CLAYS....29

Methods of Measuring Swelling
a. Measurement in the Consolidation Machine....29
b. The California Method....30
c. The California Bearing Ratio Method....32
d. The Method of Finn and Strom....32
Stability Analyses in Expansive Clay Slopes....34

CHAPTER V. THE RELATIONSHIP BETWEEN SWELLING PRESSURE AND EFFECTIVE STRESS....36

Theoretical Considerations....36
The Practical Approach....39

CHAPTER VI. THE SAMPLE, APPARATUS, AND TEST PROCEDURE...40

The Sample....40
The Apparatus....42
Test Procedure
a. Standard Tests -- Standard Cell....51
b. Constant Volume Tests....51
c. Standard Tests -- Steel Cell....54
d. Rate of strain and failure criterion....55

CHAPTER VII. DISCUSSION AND EVALUATION OF TEST RESULTS....57

Sources of Error
a. Hydrostatic Uplift on the Piston of the Steel Cell....57
b. Errors in the Measurement of Swelling Pressure
1. Vertical Swelling Pressure....58
2. Horizontal Swelling Pressure....62
c. Errors in the Measurement of Pore Pressure....63
Results obtained in the Standard Tests -- Standard Cell....65
Results obtained in the Standard Tests -- Steel Cell....70
Results obtained in the Constant Volume Tests
Swelling Pressure Included....74

PAGE

CHAPTER VIII. CONCLUSIONS AND RECOMMENDATIONS..... 85

BIBLIOGRAPHY..... 87

APPENDIX I DESIGN OF THE STEEL TRIAXIAL CELL..... 93

APPENDIX II DESIGN OF THE APPARATUS FOR MEASURING SWELLING PRESSURE IN A VERTICAL DIRECTION.....104

APPENDIX III DETAILED TEST PROCEDURE.....113
a. Preparation of Remoulded Samples.....113
b. Procedure for Constant Volume Tests.....113
c. Procedure for Standard Test -- Steel Cell..... 125
d. Sample Data Sheets and Calculations..... 126

APPENDIX IV NOTES ON SHEAR STRENGTH AND SHEAR STRENGTH TESTING OF COHESIVE SOILS.....143

The Principle of Effective Stress
a. Application to Saturated Soils..... 143
b. Application to Expansive Soils..... 145
c. Application to Unsaturated Soils..... 148
Pore Water Pressure -- Its Measurement and Importance..... 151
The Use of High Air Entry Ceramic Discs for Measuring Pore Pressure..... 154
The Measurement of Pore Air Pressure..... 156

LABORATORY DATA SHEETS -- Under separate binding.

LIST OF TABLES

TABLE		PAGE
I	Summary of Results of Stability Analyses at Dunvegan and Little Smoky River	9
II	Summary of Results of Classification and other Tests on Morrin Clay	43
III	Corrected Values of Normal Stress at Failure	
	(a) Standard Tests -- Steel Cell	59
	(b) Constant Volume Tests	59
IV	Corrected Values of Vertical Swelling Pressure	61
V	Summary of Results of Standard Triaxial Tests	71
VI	Summary of Results of Constant Volume Triaxial Tests. (Swelling Pressure included)	76
VII	Cancelled	
VIII	Relationship Between Volume Change and Pressure for Various Triaxial Cells	99
IX	Calibration of Apparatus for Measuring Swelling Pressure in the Vertical Direction (October 31, 1961)	110
X	Calibration of the Apparatus for Measuring Swelling Pressure in the Vertical Direction (March 1, 1962)	111
XI	Equations that may be Used to Express Effective Stress	144
XII	Physical Properties of Two Ceramics	155

LIST OF PLATES

PLATE		PAGE
I	Plan of Area where samples were taken	41
II	(a) Photograph -- Steel Cell mounted on support and ready to receive sample	52
	(b) Photograph -- Sample in Place and Covered with Membranes. "O" Rings about to be fitted	52
III	(a) Photograph -- Apparatus for Remoulding Samples	116
	(b) Photograph -- Cell Support and Brass Split Ring	116
IV	(a) Photograph -- Sample Consolidating in the Platform machine	121
	(b) Photograph -- Apparatus for Measuring Vertical Swelling Pressure	121
V	(a) Photograph -- The Steel Cell in the Compression Machine	124
	(b) Photograph -- The Standard Cell in the Compression Machine	124
VI	(a) Photograph -- General View of the Control Panel and Cells	142
	(b) Photograph -- The Constant Pressure Mercury Control	142
	(c) Photograph -- A Sample Consolidating in the Standard Cell	142

LIST OF FIGURES

FIGURE PAGE

1. Typical Landslide Topography in the Dunvegan Creek Area..... 4
2. Site Plan of Dunvegan Hill..... 6
3. Site Plan of Little Smoky Hill..... 6
4. The Diffuse Double Layer and Clay Micelle.....18
5. The Null Indicator (N.G.I.).....44
6. Schematic Diagram of the Control Panel for Triaxial Tests..... 45
7. Plot of Void Ratio vs. Log. Pressure (Remoulded Morrin Clay)... 66
8. Plot of Water Content vs. Log. Compressive Strength (Remoulded Morrin Clay)..... 67
9. Plot of Mohr Circles -- Total Stress. (Remoulded Morrin Clay)..... 72
   (a) Standard Tests
   (b) Constant Volume Tests. (Swelling Pressure included)
   (c) Standard Tests -- Steel Cell
10. Plot of Mohr Circles -- Effective Stress (Remoulded Morrin Clay)..... 73
   (a) Standard Tests
   (b) Constant Volume Tests (Swelling Pressure included)
   (c) Standard Tests -- Steel Cell
11. Consolidation Test Results -- Undisturbed Morrin Clay Sample A-3..... 78
12. A Comparison Between Effective Stress Envelopes. Standard Tests and Constant Volume Tests (Swelling Pressure included)...82
13. Cancelled
14. Plot of Pressure vs. Volume Change in Triaxial Cells..... 101

FIGURE PAGE

15. Plot of Temperature Change vs. Volume Change in Triaxial Cells.....102

16. Working Drawing -- The Steel Triaxial Cell.....103

17. Working Drawing -- The Pressure Measuring Apparatus.....109

18. Calibration Chart for the Pressure Measuring Apparatus..... 112

19. Working Drawing -- The Brass Split Ring and Dural Loading Post..... 117

20. Working Drawing -- The Cell Support..... 118

21. Working Drawing -- The Brass Piston Clamp..... 123

22. Working Drawing -- The Brass Piston Clamp and Standards..... 127

23. Schematic Diagram of the Self Compansating Mercury Control..... 141

24. Plot showing the Probable General Form of the Relationship Between the Parameter x and Degree of Saturation.....150

GLOSSARY OF TERMS

A	Electrical interparticle attractive pressure
Ac	The average cross-sectional area of the sample at the end of the period during which it had free access to water (sq. cm.)
Af	The average cross-sectional area of the sample at failure (sq. cm.)
Ap	The average cross-sectional area of the piston of the steel cells (sq. cm.)
ASCE	American Society of Civil Engineers
a_a	The ratio between the area of the potential shear surface which is air, and the total area
a_c and a_m	The ratio between the horizontal component of area of contact between mineral particles, and the total area of the cross section
a_s	The ratio between the horizontal projection of the area between adjacent mineral particles, through which the stress in the soil skeleton can be transmitted, and the total area.
a_u	The ratio between the horizontal projection of the area between adjacent mineral particles which are in such contact that water which can transmit hydrostatic pressure is included, and the total area.
a_w	The ratio between the area of the potential shear surface which is water, and the total area.
c	The cohesion of the soil (Kg/sq. cm.)
c'	The effective value of cohesion (Kg/sq. cm.)

E.I.C.	The Engineering Institute of Canada
e	The void ratio of the soil
micron	The one-thousandth part of a millimeter
N.G.I.	The Norwegian Geotechnical Institute
P_s	The swelling pressure of the soil. (Kg/sq.cm.)
P_{SH}	The swelling pressure of the soil measured in a horizontal direction (Kg/sq.cm.)
Psv	The swelling pressure of the soil measured in a vertical direction (Kg/sq.cm.)
P_a	The Normal stress on a potential shear surface caused by air pressure. (Kg/sq.cm.)
Q_c	Consolidated Un-drained Triaxial Test
R	Electrical interparticle repulsive pressure
S	Degree of saturation
s	Shear strength of the soil (Kg/sq.cm.)
u or P_p	Pore pressure or stress in the pore water of the soil.
u_1	Pressure in the gas or water phase of the soil
u_2	Pressure in the pore water of the soil
ϕ	The angle of internal friction of the soil (Degrees)
ϕ'	The effective angle of internal friction of the soil
σ	Total normal interparticle stress acting in the soil
σ'	Effective normal interparticle stress acting in the soil (σ -u)

σ_1 Major principal stress

σ_3 Minor principal stress

$\bar{\bar{\sigma}}$ The average vertical stress in the mineral particles at the zones of contact.

CHAPTER I

THE PROBLEM AND SCOPE OF THE RESEARCH

INTRODUCTION

The clay shales found in Northern Alberta, the Peace River block of northeastern British Columbia and the southern area of the Yukon, exhibit certain undesirable properties that have quite recently made them the object of much research and investigation.

These soils are over-consolidated clay shales formed from fine grained sedimentary deposits of clay and silt. They have a laminated or thinly stratified structure, and in general are quite stiff; however, they appear to deteriorate when they come in contact with water, especially under conditions of reduced overburden pressure.

Engineering construction, when undertaken in many of the river valleys in the above-mentioned areas, frequently produce changes in the stability conditions of the valley slopes, and trigger large earth movements. A significant peculiarity of these shales is that conventional methods of subsoil exploration do not necessarily reveal areas of potential instability, and conventional methods of stability analysis usually fail to give an adequate assessment of the stability of these slopes, either as they exist naturally, or as they may be modified by engineering works.

Geologic evidence shows that these clay shales were formed

by the compression of pre-glacial clay deposits under the weight of the continental ice sheet which covered the area during Pleistocene times. As the ice sheet melted, this compression load was reduced and eventually removed, and subsequent to this there was a further reduction of load as the top-most portion of the deposit was stripped away by erosion. The lessening of the compressive load initiated a rebound in the shale, which is still taking place today, and which has produced in the deposits, an extensive network of cracks, fissures and slickensides that now facilitate the access of water by percolation and subsoil seepage to certain layers, which absorb it and in so doing undergo a volume increase, thus speeding up the process of deterioration.

The clay shales contain a comparatively high percentage of montmorillonite clay mineral, which has the property of taking up water in its molecular structure to an extraordinary degree, provided that the moisture is available, and the soil is at a reduced overburden pressure. When these conditions are satisfied, the shale reverts to a clay of medium to high plasticity, and assumes the characteristics accompanying high overconsolidation. As the soil takes up moisture, it tends to increase in volume, and if this tendency for volume increase is prohibited, a swelling pressure results.

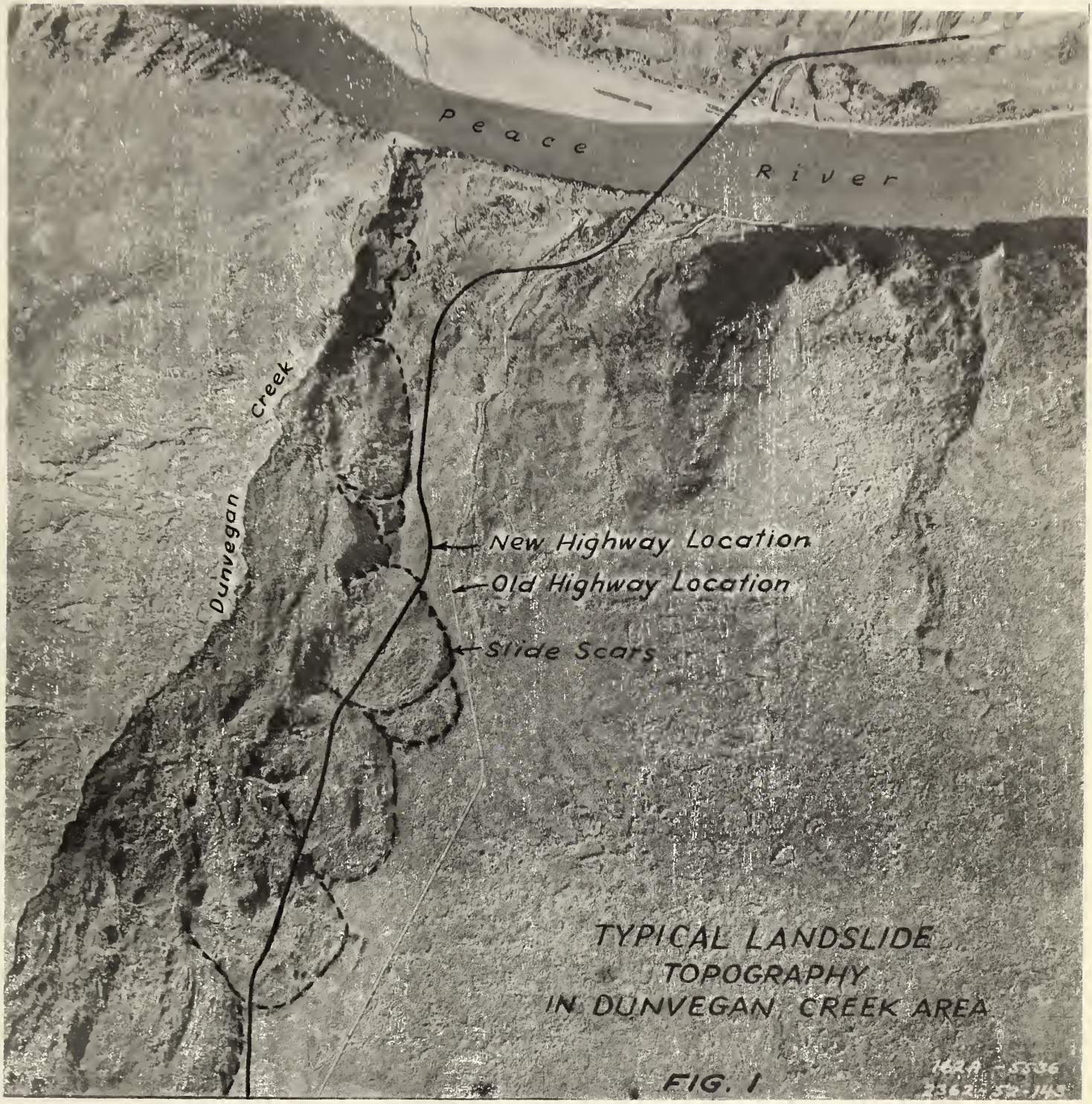
Evidence of earth movement is widespread along the valleys of rivers which have entrenched themselves in these clay shales, and in areas remote from the valleys, the sides of deep excavations made in

the shale have moved inwards, partially or completely filling the openings. Engineering works constructed in these deposits must be designed to prevent damage which may be caused by volume changes in the soil.

In the following paragraphs, a brief resume will be given of the experience encountered at two sites in the Province of Alberta, as reported by Hardy et al (1960), where it was claimed that conventional methods of stability analysis failed to give an adequate estimate of the existing conditions. Also given is a proposed modification to the Coulomb-Hvorslev equation, which the authors claim permits a more accurate assessment of the stability conditions in over-consolidated clays.

Summary of Paper by Hardy, Brooker & Curtis, 1960

In the fall of 1958 work was started on the construction of a portion of Alberta highway No. 2 which ran through the Dunvegan Valley (see Fig. 1). On this highway a new suspension bridge spans the Peace River at Dunvegan, at a point where the river valley is about seven hundred feet deep, and has an average slope of about nine degrees (Fig. 2). In order to locate a satisfactory road alignment out of the valley, it was necessary for the highway to cross old slide areas, and to have large cuts and fills. At the site at which the slide occured on May 23, 1959, the amount of fill required was about one hundred feet. About seventy feet of this fill had been placed before construction work recessed for the winter of 1958, and the opera-



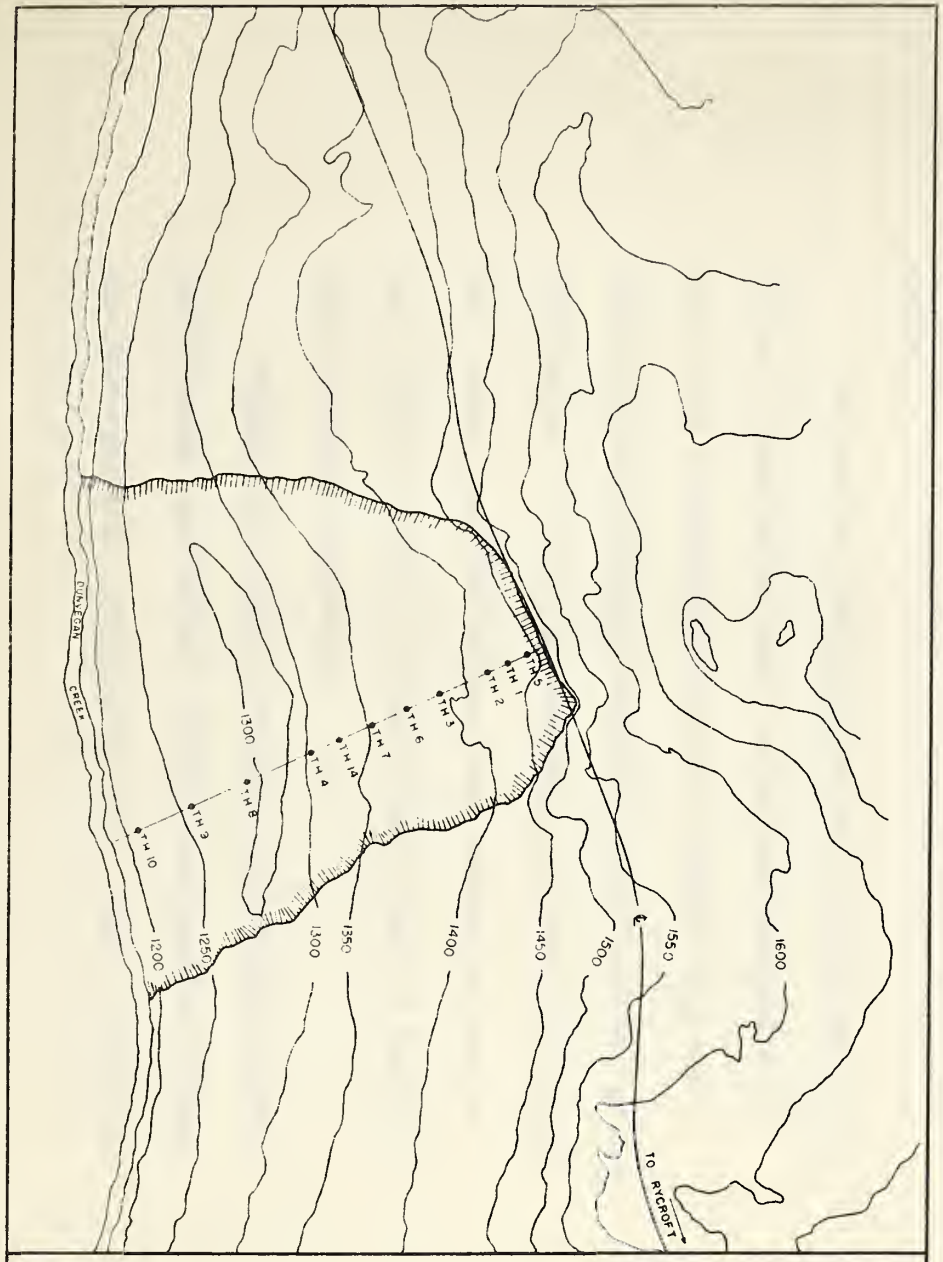
TYPICAL LANDSLIDE TOPOGRAPHY IN DUNVEGAN CREEK AREA

FIG. 1

tion was resumed when the work recommenced about May 1, 1959. On May 19, 1959 four days prior to the slide, cracks were noticed on the ground surface below the embankment, and these increased in size and number until the slide occured.

The slide itself had very little depth but was about sixteen hundred feet long. It involved between four and six million cubic yards of soil, and extended over an area of about fifty acres. Within the slide area there was an extensive network of cracks, some of which were as much as three feet wide, and twenty feet deep. The authors report that subsoil exploration carried out in the area after the slide occured, failed to reveal the cause of the instability.

The second slide occured at a bridge site on the Little Smoky River. The river valley in the area of the bridge is about 300 feet deep, and has a slope of about twelve degrees in the vicinity of the bridge, while the average slope elsewhere is about seven degrees. (Fig. 3). The West bank of the river is an old slide area, having several springs and pools caused by the ponding of surface waters. A routine subsurface exploration of the area prior to construction of the bridge showed no cause for instability.

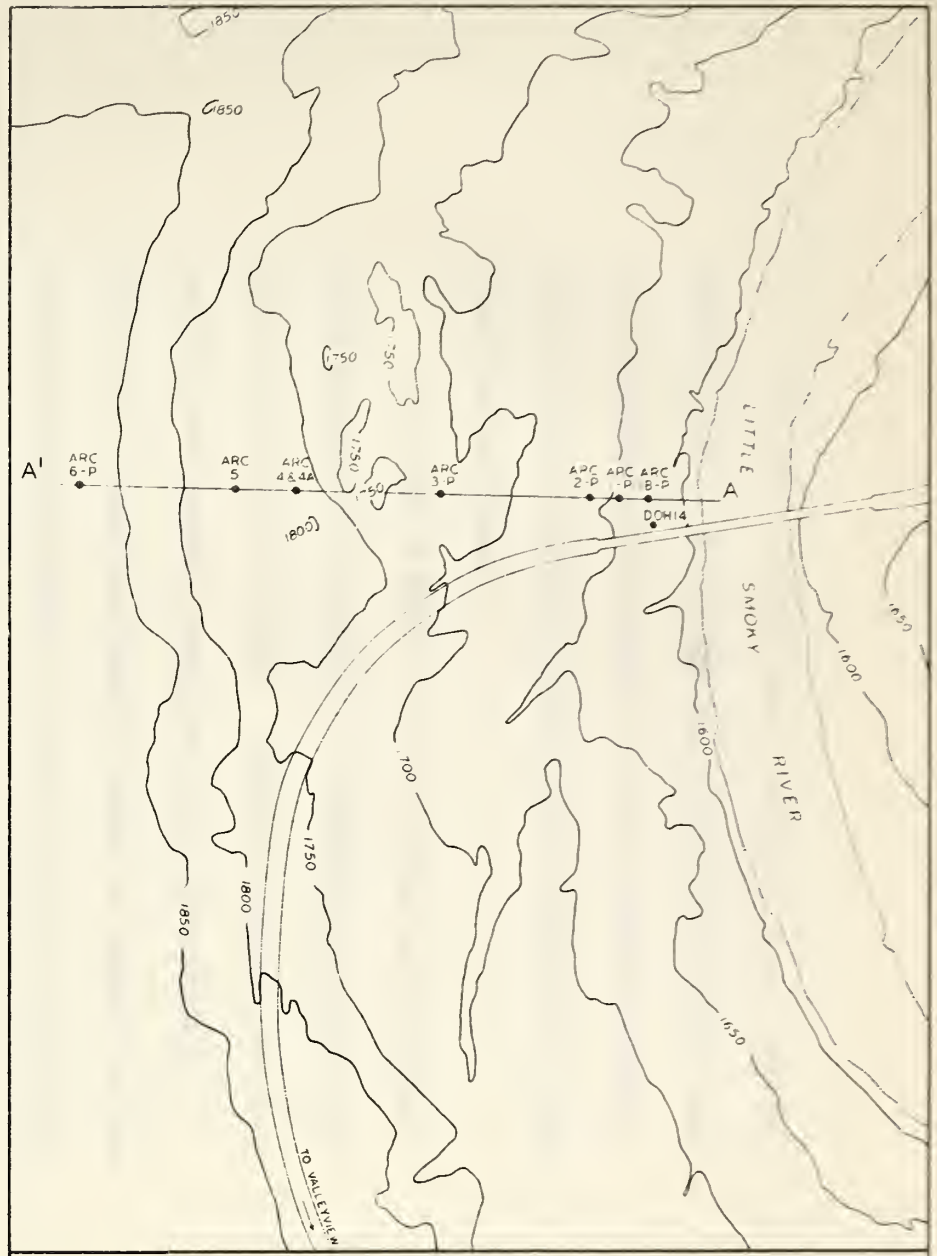


SITE PLAN
OF
DUNVEGAN HILL

Scale in Feet
200 0 200 400 600 800

Contour interval 50 feet

FIG. 2



SITE PLAN
OF
LITTLE SMOKY HILL

Scale in Feet
200 0 200 400 600 800

Contour interval 50 feet

FIG. 3

FIG 2&3

After construction was complete, it was observed that one pier of the bridge had moved appreciably causing considerable damage to the timber pile foundation. This pier was underpinned with steel H piles, but movement has continued.

In the fall of 1960 seven test holes were drilled along the line A - A' (Fig. 3), extending from the river to the top of the west bank. The holes which were located in the unstable area around the pier showed a layer of wet sand at a depth of twenty to twenty five feet below the ground surface, and the results of laboratory tests indicated that the unstable area extended to this depth.

For both the Dunvegan and the Little Smoky River slides, consolidation tests, unconfined compression tests, and consolidated undrained triaxial (Q_c) tests were run on undisturbed samples taken from the various drill holes. In each case stability analyses of the slope were made using the infinite slope method as well as the method of slices, but each analysis gave a factor of safety greater than unity.

Suggestions made by Henkel and Skempton (1955) and Skempton and DeLory (1957), were applied to both the Dunvegan and Little Smoky River slides, using the

infinite slope theory and the average depth and slope of the shear surface, but again the factor of safety was greater than unity. Table I gives a summary of the variations that were considered, and shows that in each of the two cases in which the pore pressure was considered to be unequal to zero, the magnitude required to cause instability of the slope was greater than any pore pressure recorded in the slide area. (One point to be noted here is that average values of strength were used in the stability analyses, while the actual values found in unconfined compression tests varied between 13.27 and 0.25 Kg/sq.cm.)

At the present time, the main reason given for the observed discrepancy between laboratory tests and actual field conditions has been that over-consolidation produces a dilatant structure in the soil during shear, and this produces a negative pore pressure which can cause an increase in the effective stress acting in the soil.

In order for this reasoning to be applicable to a given case, negative pore pressure should be observed during a Q_c test, but none of the tests run on the samples taken at Dunvegan or Little Smoky River showed

TABLE I

SUMMARY OF RESULTS OF STABILITY ANALYSES AT DUNVEGAN AND LITTLE SMOKY RIVER.

Assumption Used	Factor of Safety		Piezometric Level Required for F.S. = 1	
	Dunvegan	Little Smoky	Dunvegan	Little Smoky
c' $\emptyset$ fully mobilized $u = 0$	2.81	2.95	-	-
c' $\emptyset$ fully mobilized $u \neq 0$	1	1	29.0'	29.5'
$c' = 0$ $u = 0$	2.50	1.72	-	-
$c' = 0$ $u \neq 0$	1	1	11.0	-5.0'

Note: Datum taken as ground surface.

negative pore pressure. The indication here is that dilatancy does not satisfactorily explain the behaviour of the over-consolidated clays of Northern Alberta.

The authors point out that the use of Coulomb's equation

$$s = c' + (\sigma - u) \operatorname{Tan} \emptyset' \text{ --- (1)}$$

is now widely accepted as being adequate for expressing the shear strength of normally consolidated cohesive soils, but it appears that this equation should be modified in order for it to be successfully used with

over-consolidated clay soils. [Lambe (1960) has suggested that the expression for total normal stress in a cohesive soil should be modified to include the net interparticle repulsion and attraction acting in the soil. The expression proposed by Lambe for the total normal force acting across a potential shear surface is

* $\sigma = \bar{\sigma} a_m + P_a a_a + u a_w + R - A$ ---(2)]

The data on the slides at Dunvegan and Little Smoky River were examined with the object of making modifications to the Coulomb equation so that the available shear strength calculated by the use of this equation would accurately portray field conditions. This examination revealed that the value of the pore pressure "u" was the most unrealistic, in comparison with the physical conditions actually known to exist in the slide area. The known conditions gave a value of pore pressure far less than the value needed for failure conditions, and so this prompted the idea of introducing a fourth parameter into the Coulomb equation, which would be of the same nature as the pore pressure term "u".

* For definition of terms see the glossary.

Because of this the authors have suggested that the fourth parameter to be introduced in the Coulomb equation should be the swelling pressure of the soil. With this modification the Coulomb equation would take the form:-

$$s = c' + (\sigma - u - Ps) \text{ Tan } \phi' \text{ ---(3)}$$

The average swelling pressure obtained in nine consolidation tests made on samples taken from the Little Smoky River slide was 1.25 Kg/sq.cm. while the average value from four constant pressure tests made on samples from the Dunvegan slide was 1.8 Kg/sq.cm. When the stability of the slides was checked using the modified form of the Coulomb equation (3) the values of factor of safety were reported as

Dunvegan Slide	0.97
Little Smoky River Slide	0.98

In each case the magnitude of the swelling pressure needed to give a factor of safety of unity was well within the range of swelling pressure found in constant pressure consolidation tests.

Purpose and Scope of the Research.

The success achieved by the use of the swelling pressure in the analysis of the above mentioned slides has prompted further investigation into the general validity of the proposed modification of the Coulomb

equation (3).

The purpose of this investigation is to ascertain the effects of swelling on the shear strength of expansive clays. A thorough investigation of these effects would entail a great deal of time, and work, and is beyond the scope of this thesis. Because of this, the investigation has been limited to the measurement of swelling pressure in the triaxial test, and a comparison of strengths obtained by using Coulomb's equation (1) and its proposed modified form (3).

CHAPTER II

THE STRUCTURE AND BEHAVIOUR OF CLAY SOILS

The Clay Minerals

Some of the theories that have been advanced to explain the phenomenon of swelling in fine grained soils are based on the physico-chemical properties of these soils, and so an understanding of these theories demands some knowledge of the behaviour of these soils when they come in contact with water or when the cations adsorbed on the surfaces of the mineral particles are replaced.

Clay size particles are defined as all those whose diameters are small enough to allow their surface properties to be dominated by the chemistry of the material. The upper limit of size is an equivalent Stokes diameter of two microns,* and as the diameter becomes smaller the surface electric charge increases until in the colloidal range the particles have an enormous capacity for chemical activity.

The clay minerals which are foremost in abundance and importance are Kaolinite, Illite and Montmorillonite, (Grim 1940) listed in ascending order of their ability to swell, undergo replacement of the adsorbed cations, and undergo reduction in particle size when the mineral comes in contact with water. They originate mainly from the ero-

* M. I. T. Classification

sion and chemical weathering of rock, but may sometimes be formed by hydro-thermal or tectonic activity. Climate and time seem to be the predominant factors in determining the particular clay mineral found in any locality.

Identification of the kind and amount of clay minerals present in a given soil is sometimes necessary in order to correctly classify it as to its suitability for a given purpose. Several methods have been devised for this purpose, and of these the X-ray diffraction technique is the most reliable. (Grim 1958). When it is necessary to have an accurate, complete, and quantitative description of the clay minerals present in a given sample, it is generally preferable to use a combination of various methods, rather than any single one.

The Structure of Clay Soils

In considering the structure of clay soils, consideration will be given to the physical arrangment of the particles as well as to the bonding forces acting between adjacent particles. The reason for this approach is to show that structure controls the properties of the soil, and must be included in any attempt to understand the effect on these properties, caused by changes in stress conditions.

One of the earliest concepts of shear strength in cohesive soils attributed cohesion to a bond of adhesive forces acting between the adjacent mineral particles of the soil. This adhesive bond was assumed to be dependant upon the crystal structure of the mineral phase and the

atomic structure of the liquid phase.

Casagrande (1932) postulated a loose honeycomb structure for clays deposited in salt water, and in agreement with the earlier concepts, attributed the strength of the soil to the molecular forces acting at the points of contact of the large soil grains. He showed also that remoulding the soil would disturb this original structure and bond, and lead to a loss of strength.

As time went on theories were advanced which explained the bond between soil grains in terms of the colloidal chemistry of the soil. Lambe (1953) proposed a cardhouse structural arrangment of the particles, held together by electro-chemical bonding forces, whose strength depends on the kind and amount of exchangeable cations adsorbed on the mineral surfaces.

In a later paper, Lambe (1958a) published the results of studies carried out on a compacted clay, and here he divided these electro-chemical bonding forces into three categories:-

1. Primary Valence Bonds. These are described as the strongest forces holding the atoms together in the basic mineral units, and they may be subdivided into Ionic bonds, Covalent bonds, and Herter-polar bonds, all of which depend upon the exchange of electrons between the atoms in the crystal lattice of the mineral. To a lesser extent, electrostatic attraction and repulsion of electrically charged particles may be considered as Ionic bonds.

2. Hydrogen bonds. These bonding forces occur when an atom of hydrogen is quite strongly attached to two other atoms. Hydrogen bonds are not as strong as primary valence bonds, but both are large enough to withstand the stresses normally applied to the soil by engineers.

3. Secondary valence bonds or van der Waals' Forces. These are attractive forces which originate from electric moments existing between the basic units of the soil. They are capable of acting over relatively large distances, but are weaker than either Hydrogen or Primary valence bonds. Secondary valence bonds are greatly influenced by stresses applied to the soil.

The electro-chemical concept of bonding forces supplements the theory of atomic and molecular bonding forces, so that the overall picture shows that loss in strength of clay soils may be brought about by breaking the molecular or electro-chemical bonds. Increasing the stresses on the soil, remoulding, or changing the kind and amount of exchangeable cations present, are methods of breaking these bonds, and the last mentioned appears to be quite important since it affects the strongest (electro-chemical) bonds.

The Behaviour of Clay Soils

a. The Influence of Adsorbed Ions. The mineral particles of clay soils generally have a net negative electric charge which is balanced by the positive charge of the exchangeable cations present in the soil.

In the presence of water, these cations (along with a smaller number of anions) gather around the particle in such a manner that their concentration is greatest near the particle, and decrease with increasing distance from it. The collection of cations surrounding the clay particle is called the "Diffuse Double Layer" and the clay particle plus the double layer is called the "Micelle" (See Fig. 4).

The entire double layer acts as an electric field in which the water present is attracted to the electrically charged soil or to the cations, which in turn are attracted to the soil. These attractive forces are relatively weak, and are readily disrupted by a change in the interparticle spacing, or in the intergranular stress in the soil.

Different cations have varying water holding capacities, so that cation exchange in soils is accompanied by a change in the amount of water in the double layer, and this change results in a change of strength.

b. The Nature and Importance of Water. The state in which water exists in the soil, and the magnitude of the bond between water molecules and soil mineral particles are still in doubt, but it is generally accepted that water in clay soils exists in three states:-

(1) Adsorbed Water or water which is held firmly to the clay particle surfaces. This water is known to have properties unlike ordinary water, and it is also known to be oriented in some fashion around the particle and for some distance out from the surface (Grim 1958). The force with which the oriented water is held to the mineral particle decreases

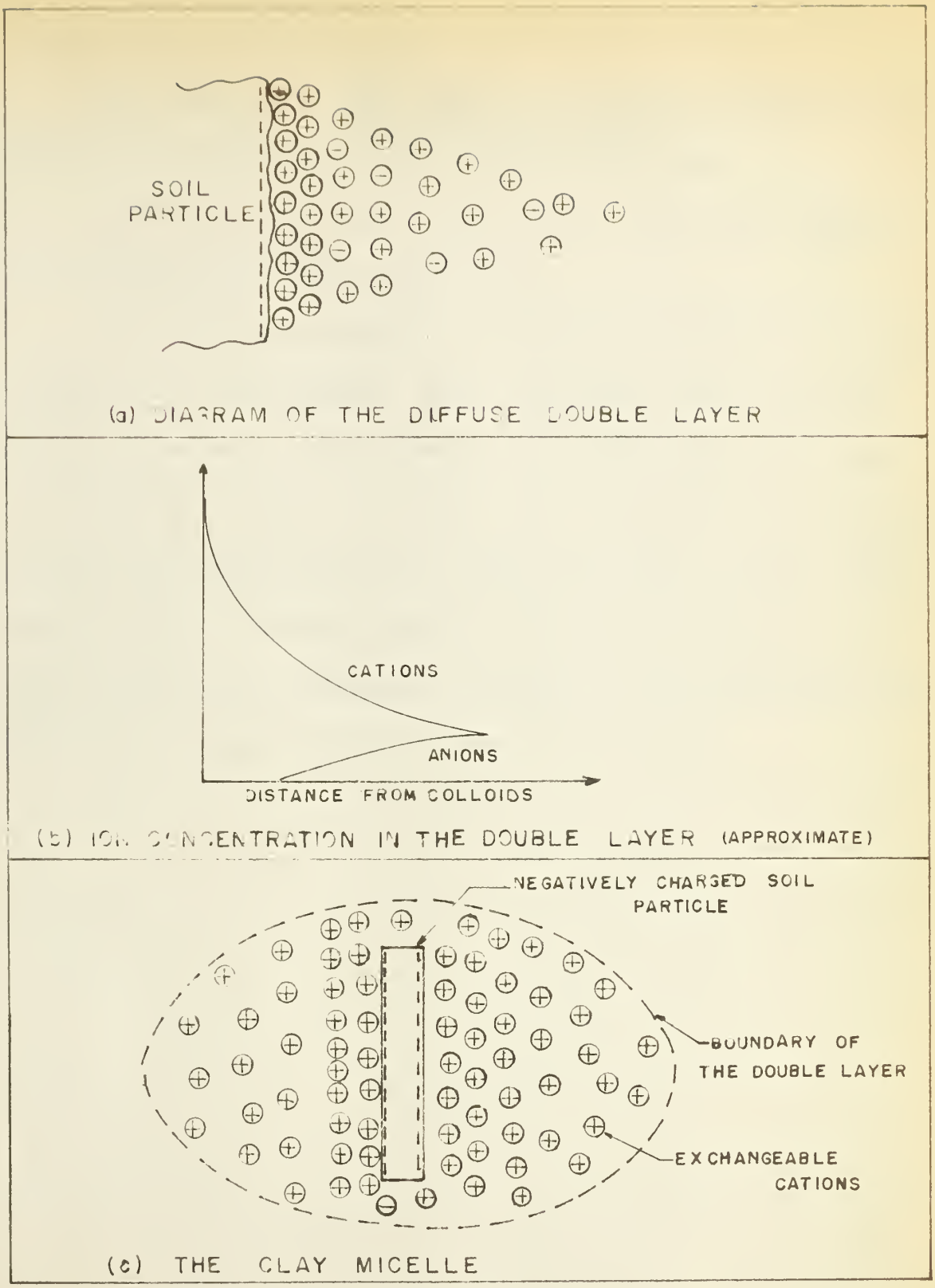


FIG. 4

THE DIFFUSE DOUBLE LAYER AND CLAY MICELLE

with increasing distance from it.

(2) Double Layer Water. This is all the water attracted to the soil particles which has properties unlike normal water.

(3) Free Water or water that is present in the voids of the soil but is not attracted to the soil particles.

The contribution made by adsorbed water to the shear strength of soils is still being debated. Some researchers believe that water bonds are capable of resisting shear in soils (Terzaghi, 1941), Rosenquist (1959), while others believe that molecular and electrical attraction between the mineral particles are mainly responsible. (Lambe 1958b), Michaels (1959). This latter group is of the opinion that water in clay soils controls shear strength only in so far as it exerts an influence on the particle arrangment and assists in cation exchange.

CHAPTER III

MECHANISMS OF SWELLING

Definition and Early Concepts of Swelling

Swelling in soils is a phenomenon that has been observed, utilized and studied long before the advent of soil mechanics as a science. The swelling of both organic and inorganic material has been investigated by botanists and agronomists for over thirty years and it is from a botanist (Katz 1933) that we get swelling in a soil defined as the process in which the soil absorbs liquid and at the same time retains its microscopic homogeneity, enlarges its dimensions and has its cohesion diminished.

The early investigators concluded from the results of their experiments that swelling in a soil depended mainly on its physico-chemical properties, and they made it quite clear that there was a difference between the capillary intake of water, and osmotic imbibition. Falconer and Mattson (1933) showed that the water of osmotic imbibition in a soil differs from hygroscopic moisture, in that it is held by forces of electrical attraction that are much weaker than the molecular forces which hold hygroscopic water, it exhibits no measurable heat of wetting, and it evaporates in ordinary air.

On the basis of these observations the mechanism of swelling was

explained from the physico-chemical viewpoint, and this explanation still has much merit. In recent years, several theories have been advanced which attempt to explain the mechanism of swelling by a consideration of the physical properties of the soil, and of these the concept of capillary tension release (Locked Pressure Theory) predominates. A discussion of several of the current theories now follows.

Theories Based on the Physico-Chemical Properties of the Soil

a. The Osmotic Pressure Theory: Osmosis is the phenomenon in which a solvent will pass in one direction only through a semi-permeable membrane from the pure solvent, or from a solution having a low concentration of the solute, into a solution having a high concentration of the solute. The passage of the solvent continues until the concentration of the solute on either sides of the semi-permeable membrane is equal. If at any time during the process it is desired to stop the migration of the solvent, a pressure must be applied to the stronger solution, and the magnitude of the required pressure is known as the osmotic pressure.

The analogy when applied to clay soils uses the clay micelle as an osmotic cell in which the electric field around the double layer acts as the semi-permeable membrane, the water and exchangeable cations in the double layer represents the solution of high concentration, and the pore water outside the double layer represents the solution of low concentration. In a closely packed soil, the double layers of the particles

overlap and create a region of high ion concentration into which water from the soil voids migrate causing a reduction in this ion concentration. In the central plane between two adjacent mineral particles, the electrical force is zero and so the water molecules in this region give rise to a hydrostatic pressure which causes an increase in interparticle spacing, or swelling.

The passage of water into the double layer can be stopped by the application of a pressure to the soil, and the magnitude of this pressure is called the swelling pressure. Swelling pressure then is the osmotic pressure in the double layer of the soil.

Swelling in a soil is greatly influenced by the crystal structure of the soil, the surface area of the soil particles, and the kind and amount of exchangeable cations present. Sodium ions are more easily dissociated than calcium or potassium ions, and so sodium saturated clays give higher swelling pressure or larger volume changes than clays having calcium or potassium as the exchangeable cations. From the point of view of crystal structure, Montmorillonite will show a greater increase in volume than either Kaolinite or Illite, because it has the ability to split easily along the basal planes of the crystal, and because of its capability to hold a larger amount of exchangeable cations.

The Osmotic Pressure theory of swelling explains that the water of osmotic imbibition causes a hydrostatic repulsive force to act in the

double layer between the soil particles, which is in excess of the pore pressure that will be measured in the soil if a piezometer was installed. This excess pressure is counterbalanced by the effective stress acting in the soil and the forces of attraction acting between the particles.

The application of the Osmotic Pressure theory of swelling seems to be limited to clays having an inter-particle spacing greater than ten to twenty Angstrom units (Ladd 1960). When the inter-particle spacing is less, there may be a significant change in the inter-relationship between the forces acting in the soil, along with an imperfect development of the double layers, resulting in the absence or reduction of osmotic pressure.

b. The Auto-Disintegration Theory. (Sideri 1936) attributed swelling in soils to the physico-chemical properties of the soil, and recognized two mechanisms by which the phenomenon may occur. The first is based on the osmotic imbibition of water and is very similar to the concept outlined above, while the second is characterized by a complete reconstruction of the inner structure of the clay micelle. This second type of swelling is described by Puri (1939) as being the result of auto-disintegration, a process by which colloids go into suspension without being shaken.

When water becomes available to a dense soil, the closely packed colloidal particles of the soil gradually go into suspension causing

a loosening of the soil, and an increase in its volume. As this destruction of the micellar structure continues, the fluid in the capillaries of the soil gradually changes to a highly viscous and very dense clay slurry. If auto-disintegration continues for a long time, all of the colloidal soil particles will form a suspension in which the larger soil grains will be dispersed.

As proof of the existence of this mechanism of swelling, Puri cites the fact that auto-disintegration is most pronounced in clays which have as their exchangeable cations Sodium and Lithium, two elements known to impart this property to soils.

Theories Based on the Physical Properties of the Soil

a. The Soil-Water Potential Theory. In a paper presented to the Twelfth Canadian Soil Mechanics Conference in December 1958, B. P. Warkentin explained the mechanism of swelling on the basis of the Osmotic Pressure theory. G. Y. Sebastyan in a discussion on Warkentin's paper objected to the Osmotic pressure theory on the grounds that the theory did not satisfactorily account for all the physical properties of clays, and proposed a Soil-Water Potential theory which he felt was capable of providing an explanation of the processes of shrinkage, swelling, cohesion and internal friction in clays.

This theory considers that each soil particle has the potential of adsorbing a given amount of water on its surface. The water molecules in the soil are under the influence of forces of molecular attraction

and repulsion which act in such a manner that the net force is one of attraction. This force decreases in magnitude as the distance from the particle increases. The state of the water films adsorbed around the soil particle will depend upon the magnitude of the net molecular force acting on them, and since the films nearer to the soil particles will be held more firmly than those further away, there will result an orientation of the water molecules with respect to the soil particle, and a difference in the physical properties of the water.

When a soil whose water holding potential is different from zero comes in contact with water, the particles will adsorb water, those nearest the source satisfying their thirst first, and succeeding particles in order. As the particles adsorb water, their unsatisfied water potential may exceed the magnitude of the other external or internal force systems acting on the soil, and the particles may become separated by the adsorbed water films, resulting in a volume increase or swelling.

It will be pointed out here that both the Osmotic Pressure theory and the Soil-Water Potential theory consider swelling to result from hydrostatic repulsive pressures arising between the planes of the clay particles, but differ in their interpretation of the source of the potential.

b. The Mechanical Concept. The validity of this theory depends

on the deformation characteristics of the mineral particles of the soil. When a soil consisting of grains of various sizes is subjected to load, the behaviour of the grains is influenced by their size; thus the large grains will fracture or undergo rotational displacement and slippage, while the fine grains, especially the flaky particles of clay size will undergo elastic deformation, imparting to the soil the ability to regain, fully or in part, its original volume when the load causing the deformation is removed.

In addition to the deformation produced when a fine-grained soil is loaded, water is squeezed out of the inter-particle voids, and when the load is released, there is a deficit in hydrostatic pressure in the voids creating a tendency for water to re-enter the soil. Such a deficit in hydrostatic pressure can only exist if the soil grains are kept apart by a mechanical force, and the presence of this force implies a direct contact between the soil particles. Direct contact between the soil particles is quite unlikely in clay soils unless they are in an advanced stage of desiccation and so it would appear that for these soils the mechanical concept does not apply.

In a soil composed of a heterogeneous mixture of grains of all sizes, it is believed that both the Mechanical theory and the Osmotic Pressure theory will apply, but a sharp distinction as to the amount of swelling contributed by either cannot be made. However it is possible to estimate the importance of either theory from a knowledge of the ion

exchange capacity of the soil or by calculation.

In commenting on the Mechanical theory of the mechanism of swelling, Bolt (1956) points out that the theory shows the existence of a direct relationship between the amount of swelling and the inter-granular stresses in the soil and so implies that there is a connection between swelling pressure and shear strength; such a relationship is not established in the Osmotic Pressure theory.

c. The Locked-Pressure Theory. Finn and Strom (1958) have presented this theory with the idea of explaining the mechanism of swelling from a purely engineering viewpoint, and have formulated it on the basis of rebound phenomena observed during triaxial and consolidation tests.

If a saturated sample is mounted in a consolidation machine and loaded, and enough time is allowed for the completion of consolidation under the applied load, the sample will undergo a reduction in height. If at this stage all the surface moisture is removed, and then the sample is unloaded, it will remain at its reduced height, save for a small amount of elastic rebound. When water is again made available to it, it will absorb the water and regain its original height.

The explanation given by Finn and Strom for this behaviour is that in the presence of surface moisture the applied load keeps the sample compressed, but when the surface moisture is removed, capillary tensions are built up in the soil as the menisci retreat into the

pores, and create a negative pressure in the water system of the soil. The access of free water to the sample relieves this negative pressure and the sample rebounds, until the negative pressure is equal to zero, or to the stresses applied to the soil.

Terzaghi (cited by Baver 1956) favoured an interpretation of the mechanism of swelling from the viewpoint of the physical properties of the soil. He attributed swelling to a combination of the effects of the surface tension of the water in the pores of the soil, and the elasticity of the solid components, and so embodied both the Locked-Pressure and the Mechanical theories.

From the variety of concepts that have been proposed to explain the mechanism of swelling, we become aware that no simple answer has yet been accepted for the explanation of the phenomenon. All the theories agree that water is of foremost importance, and that swelling is due to the adsorption of water into the soil, but the source of the force that draws the water is still being debated, with the adherents of each theory producing proof of the validity of their arguments.

There is the possibility that swelling may result from one or a combination of several of the proposed mechanisms, but the Osmotic Pressure theory, and the physical concept of capillary tension release (Locked-Pressure Theory) appear to be the most highly favoured explanations of the phenomenon.

CHAPTER IV

THE SWELLING PRESSURE OF EXPANSIVE CLAYS

Methods of Measuring Swelling

When engineering construction is to be carried out on a swelling soil, it sometimes becomes necessary to know the magnitude of volume or pressure change which can be expected under the conditions of load that would exist at any given stage. Several methods have been proposed by which reasonable estimates of the swelling pressure may be made, and the accuracy of the results depends upon the closeness of agreement between laboratory and field conditions.

Four of the principal methods now employed to measure swelling are outlined below with comments on their advantages or drawbacks, and on their agreement with the theories of swelling outlined in Chapter III.

a. Measurement in the Consolidation Machine. Two methods of measuring swelling pressure in the consolidation machine were described by Hardy (1957). In one case, the sample is placed in the machine, subjected to a very small load, and allowed to absorb water through both faces. When it has imbibed as much water as it can, the sample is loaded and the consolidation test is performed in the conventional manner. The load required to reduce the sample to its original void ratio is found from the e-log p curve, and is called the swelling pressure

of the soil.

In the other case the the sample is placed in the consolidation machine and allowed to absorb water through both faces. Volume change is prevented by the addition of small increments of load, as the sample shows a tendency to swell. When the sample shows no further tendency for volume increase, it is loaded and the consolidation test is performed. The swelling pressure is the load required to initiate volume reduction in the sample.

When the above procedures are used on similar samples, the second method in which the volume of the sample is kept constant generally gives lower values of swelling pressure. This observation is in agreement with the Osmotic Pressure theory of swelling. In the first method in which the sample is allowed to swell, pure (distilled) water enters the voids and dilutes the electrolyte concentration there, thus increasing the difference between the concentration of the fluid in the voids, and in the double layer, and so increasing the osmotic pressure. This dilution of the electrolyte concentration in the voids is restricted in the second method, and so observed swelling pressures are lower.

b. The California method. This method employs the Hveem stabilometer, and is used extensively in highway work. A cylindrical specimen, four inches in diameter, and two and one half inches high, is prepared by a kneading type compactor, subjected to pressure high enough to cause water to be exuded from the soil, and then allowed to

absorb water from one face only, for sixteen to twenty hours, while it is still in the steel mould. The lateral restraint provided by the steel mould causes the sample to expand in the vertical direction only, against a device which measures the expansion force that is developed. The drawbacks of this method are:-

1. The sample is subjected to pressure high enough to cause exudation of water from its pores. This leads to the existence of negative pore pressure in the soil so that when the test is made higher values of swelling pressure are obtained. (Locked-Pressure Theory).

2. The sample is allowed access to water from the top only, and this leads to lower values of swelling pressure than will be obtained if the sample was allowed access to water from both faces. (Osmotic Pressure Theory. Dilution of the electrolyte concentration in the voids is greater at the top of the sample than at the bottom.)

3. The dimensions of the specimen can cause the development of friction forces of appreciable magnitude which could also lead to a reduced value of swelling pressure.

4. Swelling pressures of greater magnitude would be obtained if the time of soaking was increased above the sixteen to twenty hours presently allowed. (Osmotic Pressure theory. Electrolyte concentration in the voids may be further diluted by longer soaking time.)

Finn and Strom (1958) hold that the California method does not measure the absolute value of the swelling pressure of the soil, but

provides an empirical index of its value, which when used in the empirical California design procedure, gives structurally adequate pavements.

c. The California Bearing Ratio (CBR) method. The CBR method measures the amount of swell in the soil, and is part of the CBR method of roadway design, which was originally developed by the California Division of Highways, but is now extensively used, with some modifications, by the Corps of Engineers U. S. Army.

In this method the specimen is compacted in a cylindrical steel mould, and is about six inches in diameter and four and one half inches in height. Mould and specimen are then immersed in water, and the specimen is allowed to soak up water through both faces for four days, during which time the anticipated pressure of the pavement on the subgrade is simulated by placing surcharge weights on the sample, and the amount of expansion under this expected load is found by observation on an extensometer dial.

The CBR method eliminates the drawbacks of the California method listed under 1, 2 and 4 above, and reasoning from the Osmotic Pressure theory, it should give reliable results, provided that the surcharge load is correctly estimated, and that the conditions for cation exchange in the field are approached in the laboratory.

d. The method of Finn and Strom. This method was proposed

by Finn and Strom (1958) with the intention of providing a rapid means of measuring the absolute value of the swelling pressure of soils. It is designed to eliminate the errors present in the California method, and to provide results which are as accurate as those obtained in the consolidation machine.

The sample used is cylindrical in shape, four inches in diameter, and two and one half inches in height.* It is enclosed in a top hat shaped rubber membrane, and is sealed in a pressure chamber filled with water. Access of water to the sample is by means of a porous stone at the bottom, and the required confining pressure is applied through the water surrounding the sample. As the sample absorbs water and tends to increase in volume, the pressure in the confining fluid increases, and its magnitude is recorded on a gauge provided for the purpose.

In this method axial and lateral restraining pressures are equal, and so the test conditions are very unlike the conditions present in a large soil mass, in which the soil is rigidly confined except at the boundaries. This method has an advantage over the others if the soil to be tested comes from an area in which lateral restraint is lacking and horizontal and vertical expansion are equal.

The advantages of the method of Finn and Strom are the small

*Samples one and one half inches high and of equal diameter are also used.

amount of side friction to be overcome as the sample moves against the rubber membrane, and the small amount of volume change allowed.*

Each of the above methods has its advantages depending on the problem at hand, but except for the last, (the shortcomings of which were earlier pointed out) none are designed to measure swelling pressure in a horizontal direction. If the modified form of Coulomb's equation is to be used, it will be necessary to devise some means of providing measurements in both the horizontal and the vertical directions.

Stability Analyses in Expansive Clay Slopes

Values of swelling pressure obtained by means of one of the methods outlined above are quite often used in the safe design of buildings, highways and hydraulic structures which are to be constructed on expansive clay, but there is no known method of incorporating similar values in the procedure for determining the stability of slopes in the same soil.

In Chapter II it was stated that a change in moisture content or the replacement of the exchangeable cations in the soil may lead to a loss of strength, and it is possible that these changes may be the cause of unstable conditions occuring in slopes in expansive clay. Holtz (1959)

*The method of Finn and Strom allows 0.6% volume change at a pressure of 7.5 p.s.i. while the California method allows 6.0% change at the same pressure.

discusses this problem, and advocates that careful sampling, and testing under the conditions of loading and moisture content that would be met in practice, are prime requisites for achieving a reliable estimate of shear strength.

Experience has shown that even the most careful methods of sampling and testing are inadequate, and that checks made on actual slides have failed to yield a solution. Lambe and Whitman (1959) have suggested that the answer to the problem may be in the use of an entirely new concept. This suggestion is closely parallelled by the proposed modification of Coulomb's equation (equation 3) which embodies a swelling pressure term, a variable dependent upon the kind and amount of exchangeable cations and the crystal structure of the soil. (page 22). These two factors are also known to bear a direct relationship to the shear strength, so that the addition of the swelling pressure term to the Coulomb equation may be the means of allowing this expression to reflect the possible changes that may occur in the soil.

CHAPTER V

THE RELATIONSHIP BETWEEN SWELLING PRESSURE AND EFFECTIVE STRESS

Theoretical Considerations

In Chapter I it was stated that the purpose of this investigation was to try to ascertain the effect of swelling on the shear strength of expansive clays, and it was also stated that this will be done by making a comparison between strengths obtained by the use of the general and proposed modified form of Coulomb's equation (Equations 1 and 3).

Before the proposed modified form of Coulomb's equation is used, it is desirable to determine whether the swelling pressure acts in a manner that will increase or decrease effective stress. Hardy et al (1960) have claimed, by a consideration of the tension forces acting in the water films, that the term carries a negative sign, and here a further examination will be made on the basis of a literature review.

A full understanding of the behaviour of a cohesive soil under conditions that would induce swelling can best be obtained by a study of soil physics, and the Osmotic Pressure theory (Ladd 1960, Hemwall and Low 1956, Bolt 1956) points out that a difference between the electrolyte concentration in the double layer, and in the free water of the soil is responsible for the osmotic pressure which causes water to flow from the region of low electrolyte concentration in the pores, to

the region of higher electrolyte concentration in the double layer. Theoretically this flow would continue until the osmotic pressure in the double layer is balanced by the overburden pressure, or until equilibrium is established between the osmotic pressure and the interparticle forces.

To accomodate the water that flows into the double layer, the soil grains are forced apart, and this is seen as a volume increase in the soil mass. If the stresses acting on the soil are large enough to prevent this volume increase, a swelling pressure is developed, and what is regarded as osmotic pressure can be considered to act in a manner that will reduce effective stress.

On the other hand osmotic imbibition may be considered to cause an increase in the adsorbed water films of the soil particles, so that swelling pressure may be regarded as resulting from closer interaction of the adsorbed films of adjacent particles. From this viewpoint what is regarded as osmotic pressure can be considered to act in a manner that will increase effective stress. A further point to be considered is whether or not these adsorbed water films are able to resist shearing stresses.

Lambe (1960) has proposed that the stress acting in a cohesive soil may be expressed as :-

$$\sigma = \bar{\sigma} a_m + P_a a_w + u a_w + R - A \quad ---(2)$$

In presenting equation (2), Lambe claims that considerable amount of

experimental data points to its validity, but admits that the terms R and A cannot be measured directly.

Equation (2) expresses the conditions for equilibrium in the soil, and Lambe reports that experiments have proved that by changing the value of the interparticle repulsive force R, other terms of the expression undergo changes, until equilibrium is restored. In one example given, Lambe shows that an increase in the salt concentration of the pore fluid caused a volume decrease in the sample, and this he attributed to a reduction in R. This volume decrease was of a large enough magnitude to re-establish equilibrium conditions in the sample.

For the case in which a soil exhibits a swelling pressure we have the reverse of the above situation. There is a tendency for volume increase due to an increase in the term (R - A), and in order to maintain equilibrium while maintaining constant volume in the soil there must be a change in one or more of the other terms of equation (2).

However since the terms R and A cannot be measured, and since it is not known how these terms are affected by changes in stress applied to the soil, Lambe's equation (2) leaves room for doubt.

At the present time it must be concluded that there is no conclusive proof that can be advanced to resolve the controversy, so that it cannot be definitely stated that swelling pressure acts as an effective stress, or as a neutral stress.

The Practical Approach.

An attempt will be made to test the validity of the proposed amendment to the Coulomb equation by carrying out two series of tests on an expansive soil, and making a comparison between the Mohr strength envelopes obtained in each case. One series will be standard consolidated undrained triaxial tests with pore pressure measurements, carried out according to the procedure described in Bishop and Henkel(1957), while the other series will be conducted under similar conditions, with the exception that during the consolidation period no volume increase will be allowed in the sample, and measurements will be made of the swelling pressure developed in both the horizontal and vertical directions. When equilibrium has been established, the sample will be sheared as in the consolidated undrained test. It is proposed that the first series of tests be referred to as Standard Tests, and the second series be referred to as Constant Volume Tests.

Using this procedure, it is hoped that a study can be made on the basis of effective stresses. It is expected that due to the preferred parallel arrangment of the soil particles, swelling pressure in the vertical direction will be greater than in the horizontal direction, so that the Mohr circles obtained from the Constant Volume series, would be of smaller radii than the circles obtained in the Standard Tests provided that swelling pressure acts in a manner to reduce effective stress.

CHAPTER VI

THE SAMPLE, APPARATUS, AND TEST PROCEDURE

The Sample

The soil used in this investigation was taken from the area near Morrin Alberta, and is a sedimentary deposit of post-glacial Lake Drumheller, the center of which was located in the area between Drumheller, Morrin and Carbon. (Wyatt and Newton 1943).

The deposits in this lake bottom are a uniform clay, eight to ten feet thick, high in lime content, and of a gray-black colour. The soil survey of the area reports that in general, forty per cent of the soil is less than two microns in diameter, and highways built in this area have had their surfaces disrupted due to the entrance of water into the sub-grade. This indicated the development of a reasonably high swelling pressure when the soil comes in contact with water, but is kept at constant pressure.

Plate I shows the location where the Shelby tube samples were taken. These samples showed nugget structure up to a depth of five to six feet, and the amount of sand present showed a marked increase for depths below eleven feet. One sample recovered from a depth of sixteen feet consisted mainly of fine sand, with very little clay. Traces of lime were noticeable in the sample, and there was very

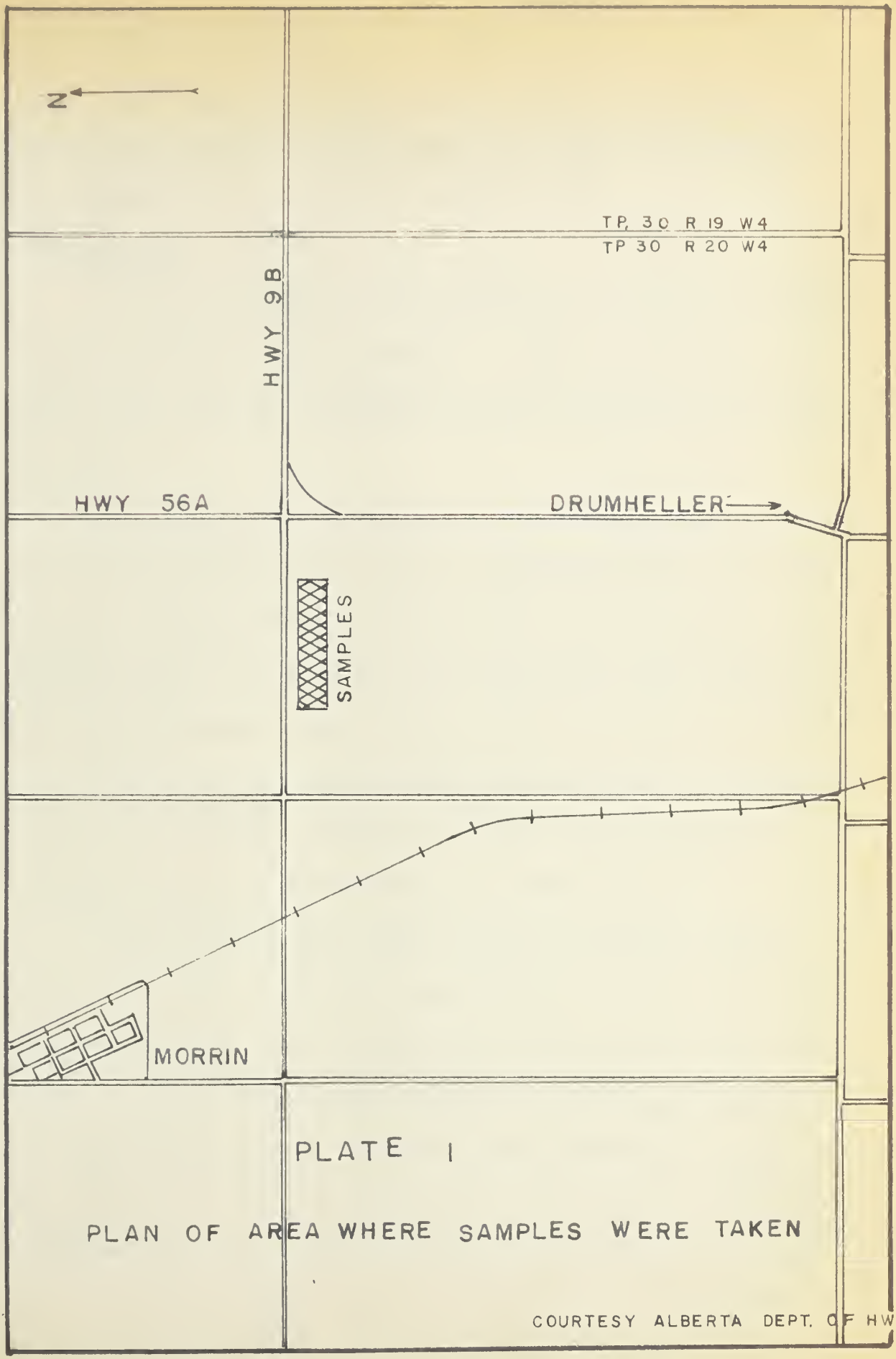


PLATE 1

PLAN OF AREA WHERE SAMPLES WERE TAKEN

COURTESY ALBERTA DEPT. OF HWYS.

little evidence of fissuring.

Table 2 gives a summary of the results of classification and other tests carried out on the soil. Sample preparation is described in Appendix III.

The Apparatus

All specimens tested had a diameter of 1.4 inches, and the apparatus used was standard equipment, except for the constant volume tests, which required the design of a special cell. (See Fig. 16, Appendix I).

For the standard tests, Leonard Farnell* cells were used, and the confining pressure in all tests was supplied by self-compensating mercury controls (Bishop and Henkel 1957). The compression part of all tests was carried out in a strain controlled machine made by Leonard Farnell (Model 301/T .124), and the null indicator used for pore pressure measurements was the type developed by the Norwegian Geotechnical Institute, a diagram of which is shown in Fig. 5. A schematic drawing of the control panel is shown in Fig. 6.

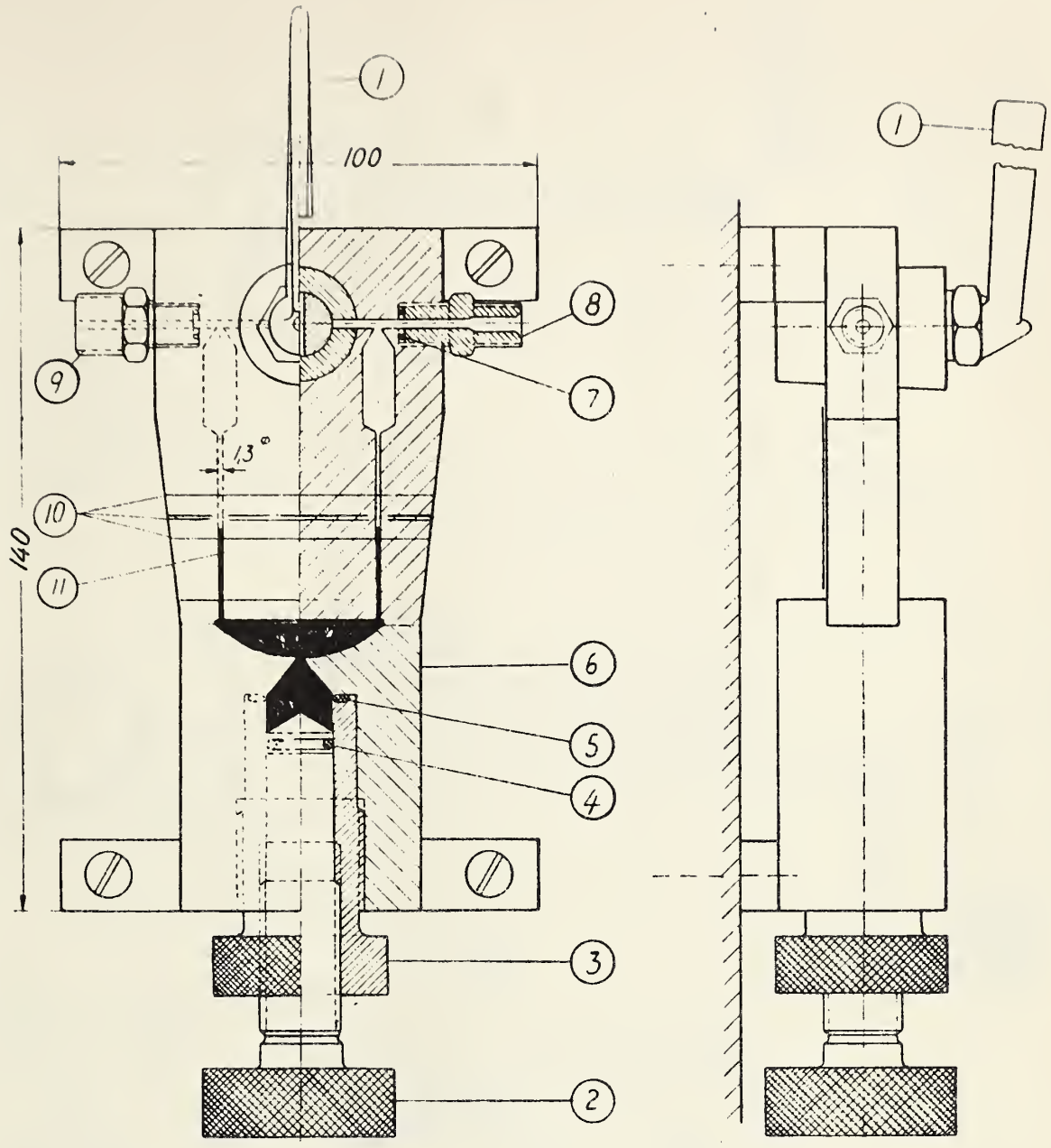
The constant volume tests required the design of a special cell, in which one could make measurements of swelling pressure in both the horizontal and vertical direction, and which would allow very little, or preferably no volume change in the sample. Since the anticipated volume changes were of the order of one tenth of a cubic

*Leonard Farnell & Co. Ltd. North Mymms, Hatfield, Hertfordshire England.

TABLE II

SUMMARY OF RESULTS OF CLASSIFICATION AND OTHER TESTS ON MORRIN CLAY

	No. of Tests	High Value	Low Value	Average Value	Remarks
Specific Gravity	9	2.89	2.83	2.86	
Natural Moisture Content	15	40.3	32.9	36.5	Depth Approx 7'
	6	41.2	31.1	34.9	Depth Approx 9'
Atterberg Limits					
Liquid Limit	5	104.2	87.5	97.1	
Plastic Limit	5	37.4	27.4	33.3	
Plasticity Index	5	73.4	56.6	64.5	
Shrinkage Limit	5	17.3	11.3	14.5	
Grain Size Distribution					
% Sand Sizes	3	16	12	14	M.I.T. classification
% Silt Sizes	3	18	14	16	
% Clay Sizes	3	70	70	70	
Unconfined Compressive Strength Kg/sq. cm.	3	1.15	0.92	1.08	
Consolidation Tests					From Undisturbed Samples
Compressive Index	4	0.354	0.168	0.28	
Swelling Pressure Kg/sq.cm.	4	0.97	0.68	0.80	Constant Void Ratio
Swelling Pressure Kg/sq.cm.	1			1.45	Constant Pressure
Preconsolidation Load Kg/sq.cm.	1			0.65	
Clay Minerals					
%Montmorillonite				53	Rough Approximation only.
% Illite				32	
% Kaolinite				15	X-ray Diffraction technique



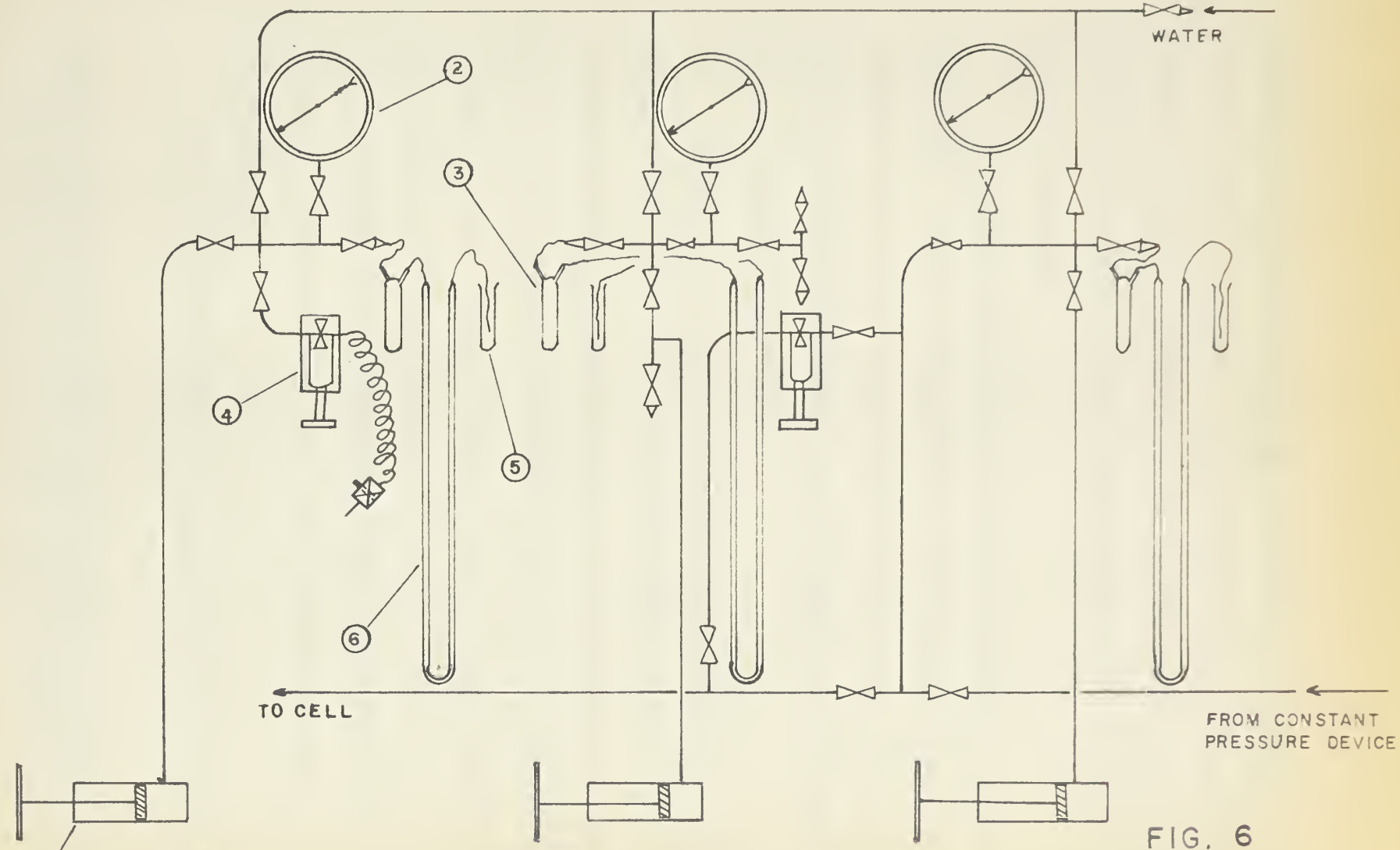
1. Klinger valve.
2. Adjusting knob.
3. Tightening screw.
4. Angus Gaco ring R 108.
5. Rubber packing.
6. Housing.
7. Angus Gaco ring R 102
8. Coupling for copper tubing.
9. Connection.
10. Pressure measuring system.
11. Mercury.

Lengths in millimetres.

FIG. 5

NULL INDICATOR (N.G.I.)

(Taken from N. G. I. Publication No. 21.)



KLINGER VALVE. AB 10.

(1) SCREW CONTROL

(2) BOURDON GAUGE

(3) MERCURY TRAP

METAL PIPES - 1/4" I.D. COPPER

(4) PORE PRESSURE INSTRUMENT

(5) OVERFLOW TRAP

(6) MERCURY MANOMETER

FLEXIBLE CONNECTIONS 1/8" O.D. PLASTIC TUBING

FIG. 6

CONTROL PANEL FOR TRIAXIAL TESTS

(N.T.S.) MARCH 31, 1962

FIG 6

centimeter, two considerations came foremost in the design. Firstly the cell itself must undergo very little or no volume change when the sample is subjected to an increase in all round stress, and secondly the ratio of specimen volume to volume of fluid in the cell should be as large as possible.

In order to satisfy the first condition, a number of calculations were made, and Fig. 14 (Appendix II), shows the results as plots of pressure versus volume change. The material of the cells was lucite and steel, and the indication was that a steel cell was better suited for the purpose than a lucite cell.

To satisfy the second requirement, it was decided that the diameter of the cell should be made quite small, allowing mainly for the space needed to prevent the specimen from touching the walls of the cell, in the event that it failed by bending. The cell that was eventually designed had steel walls one quarter of an inch thick, an inside diameter of two and one half inches and a height of five inches. The total volume of this cell was 24.55 cubic inches (402 cubic centimeters) and the ratio of specimen volume to total cell volume was approximately 0.2*

Calculations showed that a very low volume change could be

* Ratio of volume of specimen to volume of cell for the Geoner cell is approximately 0.04, and for the Farnell cell approximately 0.08

expected in this cell if it was subjected to pressure as high as 10.0 Kg/sq.cm. (See Appendix I), and Fig. 15 (Appendix I) shows that the normal daily variation of two to four degrees in the laboratory temperature will not cause a volume change of appreciable magnitude.

The entire cell consisted of a steel base, steel cylindrical section and a heavy brass cap which provided a bushing for the piston. The cylindrical section was threaded into the brass cap so that these two parts acted as a single unit, and this unit was held to the base by three studs and wing nuts. Rubber "O" rings between the cylindrical section and both base and cap, provided leak proof joints.

The brass cap was made slightly conical at the top, and from this cone it had two holes connecting to the outside. These holes served the double purpose of allowing air to escape from the cell when it was being filled, and allowing oil* to be pumped into the cell after it was filled with water. This oil floating on top of the water, served as a seal, as well as a lubricant for the piston.

The steel cell was fitted with a piston 1.4 inches in diameter and twelve inches long. This piston bore directly on the upper ceramic disc** and had the rubber membranes which covered the sample attached to its lower end. The use of a piston having the same diameter

* The oil used for this purpose was Esso Teresso No. 65

**The ceramic discs used were Celloton CF26 made by Aerox Limited, Glasgow. Maximum pore size 2.9 microns.

as the diameter of the sample, and bearing directly on the upper ceremic disc had the advantage of preventing the hydrostatic pressure in the cell from acting downwards on the sample, and in this way any upward force developed by the sample would be transmitted to the piston and measured. This measured upward force was regarded as the swelling pressure in a vertical direction (Psv).

Measurement of the upward force acting on the piston had to be made without permitting any change in length of the sample, and this was accomplished by allowing the piston to push upwards at the centre of a simply supported beam,* which was maintained in a fixed position on top of the cell. A strain gauge (Budd --C12-141) was cemented at the top center of this beam, and the deflection caused by the upward force of the piston was read from a strain gauge indicator. A calibration chart was prepared and from this the upward pressure corresponding to a given deflection could be read.

With this arrangment readings were taken to the nearest five micro inches per inch of strain, which corresponded to an upward pressure of 0.07 Kg/sq.cm.

Swelling pressure in the horizontal direction was taken as the all round stress which had to be applied to maintain the sample at constant volume. Because no upward movement of the piston was per-

*Design details of the cross beam and its vertical support are given in Appendix II.

mitted, and because the cell was designed to resist small changes in volume, the only means whereby the specimen could change volume was by expelling fluid from the cell. The use of a null indicator in the line connecting the constant pressure device to the cell, assured that the condition of no volume change was satisfied, and at the same time a measurement could be made of the pressure required to maintain this condition.

The null indicator used for the above purpose was the pore pressure measuring device used with the Geonor triaxial equipment (Fig. 5), which is essentially a mercury u-tube of very fine bore (1.3 millimeter diameter). This instrument was placed in the line leading from the constant-pressure apparatus to the triaxial cell, and at the start of the test, the by-pass valve in the instrument was opened so that the same pressure acted throughout the entire system. The by-pass valve in the null indicator was then closed and both legs of the mercury u-tube remained at the same level. When water was made available to the sample and it attempted to expand, fluid was gradually forced from the triaxial cell, causing a depression of one leg of the u-tube, and a corresponding elevation of the other. To counter balance this movement, the pressure on the elevated leg of the u-tube was increased by increasing the pressure on the constant pressure device, until the mercury level in both legs returned to the equilibrium position. The new pressure acting in the fluid of the cell was read from a Bourdon gauge or

a mercury manometer.

The very small bore of the u-tube made it possible to detect changes as small as 0.02 cubic centimeters, and so pressure increases were made before any substantial permanent volume change occured. In this manner each minute volume change was checked in a semi-continuous fashion.

Pore pressure measurement from the top porous stone was made possible by means of two holes drilled throughout the length of the piston. These holes connected with the porous stone at the bottom of the piston, and terminated at valves at the top of the piston. From the valves connection was made to the pore pressure measuring device. This arrangment allowed an attempt to be made at measuring pore pressure at the top of the sample, during the time that water was made available to it at the bottom.

Having the piston attached directly to the top of the specimen posed a special problem when loading the specimen into the cell. The solution was to make the piston long enough so that the steel cylinder and brass cap could be slid upwards until there was enough room for loading. It was mainly for this reason that the piston was made twelve inches long. A special stand (Fig. 20) was made for supporting the cylinder and cap in this elevated position, and a split brass ring (Fig. 19) was made for placing the "O" rings over the membranes.

The photographs in Plate 2 illustrate the use of the stand and the split ring.

The brass clamp shown in Fig. 21 was made to hold the piston stationary during the period between the removal of the cross beam for measuring swelling pressure, and the placing of the cell in the compression machine. The supports for the cross beam were fitted with holes for supporting this clamp.

Test Procedure

a. Standard Tests -- Standard Cell. The tests carried out in the standard cell were conducted as outlined for consolidated undrained tests in Bishop and Henkel (1957). Wool wicks placed internally, and filter paper strips placed externally, were used to shorten the drainage path and so speed up the consolidation process. Remoulded specimens were first consolidated under an all round pressure of 7.0 Kg/sq.cm., and then allowed to consolidate under the value of all round stress required for the test. This procedure was adopted to induce a swelling tendency in the specimens when the all round stress was reduced from 7.0 Kg/sq.cm.

b. Constant Volume Tests. When remoulded specimens were used, they were mounted in the cell, which was then filled with water, and a layer of oil was placed on top of the water. The loaded cell was placed on a platform type consolidation machine, and after balancing, the loading yoke was brought in contact with the loading post on top of

(a) STEEL CELL MOUNTED ON SUPPORT AND READY TO RECEIVE THE SAMPLE

(b) SAMPLE IN PLACE AND COVERED WITH MEMBRANES. "O" RINGS ABOUT TO BE FITTED.

PLATE II

the piston. The cell pressure was then increased gradually to 7.00 Kg/sq. cm. while load was simultaneously added to the machine until the vertical pressure on the specimen was also 7.00 Kg/sq.cm. Drainage of water from the sample was permitted, and the horizontal and vertical pressures were maintained until a plot of volume change versus the logarithm of time showed that 100 per cent consolidation had been reached.

When consolidation was complete, the drainage was shut off, and the vertical and horizontal stress acting on the sample were simultaneously reduced to zero. The cell was removed from the platform machine, the device for measuring swelling pressure in the vertical direction was mounted on top of the cell, and the pore pressure line was connected to the outlet on the piston. The average time required for this operation was two to three minutes.

A zero reading was then taken on the strain gauge indicator, and the confining pressure was increased to the desired initial value. The bypass valve in the null indicator was closed, drainage from the specimen was reopened, and readings were made of changes in horizontal and vertical swelling pressure, pore pressure and water volume. Observations were continued until the specimen showed no further tendency to increase either the horizontal or vertical swelling pressure.

During the night or when it was impossible to be present, the

drainage valve was closed and the bypass valve in the null indicator was opened. The assumption made here was that no swelling would take place when the water supply was shut off. This assumption appears to be valid for cases in which water was made available to the specimen for at least six hours, since it has been observed that horizontal swelling pressure attained its maximum value in six to eight hours. For cases in which the valve was closed before the six hour period, test results show a lower value of horizontal swelling pressure, indicating that some volume change took place.

When the swelling pressure in the vertical and horizontal directions showed no further increase, the brass clamp (Fig. 21) was fitted to the piston, the cross beam was removed, and the cell was placed on the compression machine. Here a load was applied to the piston which was equivalent to the upward force measured on the cross-bar, and then the clamp was removed.

The compression part of the test was carried out as in the standard test. A detailed procedure for the constant volume test is given in Appendix III.

c. Standard Tests -- Steel Cell. An additional series of tests was carried out to try and ascertain whether the test results were influenced by the design of the steel cell. The object of these tests was to try and duplicate in the steel cell, the results obtained in the standard cell.

For this test series, consolidation (at 7.00 Kg/sq.cm. as well as at the lower values of confining pressure required for the test) was carried out with the cell in the platform type consolidation machine. The vertical stress applied to the sample through the piston was made equal to the hydrostatic pressure in the confining fluid, due account being taken of the change in average area which the specimen undergoes during consolidation.

When consolidation was complete, the clamp shown in Fig. 22 was fitted to the cell to keep the piston stationary, and the cell was removed to the compression machine, where an appropriate vertical load was applied to the piston before the clamp was removed. The compression part of the test was carried out as in the standard test. A detailed procedure for this test is given in Appendix III.

d. Rate of strain and failure criterion. All tests were carried out at a rate of strain of 1% per hour, chosen to allow failure of the sample in six to eight hours, and to improve the accuracy of the pore pressure measurement. Samples tested at low values of confining pressure failed in less than six hours, but the constant rate of strain was maintained in order to allow a better comparison of the results.

The failure criterion was chosen as the maximum value of the principle stress ratio. Bishop and Henkel (1957) compare this criterion with the maximum value of deviator stress, and show that the use

of the former gives slightly higher values of c' and ϕ', while the use of the latter provide wider separation of the Mohr circles, and so a more clearly defined failure envelope.

The maximum deviator stress when used as failure criterion, gives stress circles which define a limiting envelope, but the difference in c' and ϕ' obtained by both methods is small, and usually negligible.

CHAPTER VII

DISCUSSION AND EVALUATION OF TEST RESULTS

Sources of error.

a. Hydrostatic Uplift on the Piston of the Steel Cells. The piston in the steel cell had a diameter of 1.4 inches and it was attached to the upper porous stone. The purpose of this arrangment was to prevent the confining pressure from acting downwards on the sample, and in this manner facilitate the measurement of the upward force developed by the sample as it tried to swell.

When the sample was originally mounted in the cell, it had an average cross-sectional area very close to that of the piston (10.0 sq. cms.), but after consolidation was complete under a pressure of 7.00 Kg/sq.cm. this average area was reduced by about two square centimeters. This reduction in area allowed the confining fluid to exert an upward force on the piston, and so introduced an error in the measured upward force attributed to swelling.

Before the recorded values of upward force could be used, a correction had to be applied. The exact value of this correction could not be determined, so a reasonable estimate was taken as the product between the value of all round stress acting on the sample

(σ_3) and the difference between the area of the piston and the average area of the sample.

Correction for uplift $(A_p - A_c) \times \sigma_3$

The uplift force on the piston of the steel cell also caused an error in the measurement of the normal force acting on the sample during compression. For the failure conditions, the normal force as read from a proving ring was corrected in the same manner as above using the average area of the sample at failure. Table III (a) gives the corrected values of normal stress at failure for the Standard Tests - Steel Cell, while Table III (b) gives corresponding values for the Constant Volume Tests.

b. Errors in the Measurement of Swelling Pressure.

(1) Vertical Swelling Pressure. Appendix II gives the design data for the apparatus that was used to measure swelling pressure in a vertical direction, and it is shown that the maximum load which eventually acted on the apparatus was 90.2 Kg, about three times the maximum load for which the apparatus was designed. At this maximum load it was possible for the sample to elongate .003 inches (0.1% based on a three inch sample) compared with the elongation of 0.001 inches allowed for in the design.

This apparatus was calibrated at the beginning of the tests, and a check on this calibration was made at the end of the tests, four months

TABLE III

CORRECTED VALUES OF NORMAL STRESS AT FAILURE

(a) STANDARD TESTS -- STEEL CELL						
A_f sq.cm.	8.70	8.52	9.25	8.07	8.54	8.40
$(A_p - A_f)$ sq.cm.	1.30	1.48	0.75	1.93	1.46	1.60
σ_{3f} Kg/sq.cm.	0.50	1.00	2.00	4.00	7.00	9.00
$(\sigma_{3f} \times A_f)$ Kg.	0.65	1.48	1.50	7.72	10.22	14.40
Measured Downward Force Kg.	15.90	22.90	38.00	60.02	103.75	132.40
Net Downward Force Kg.	15.25	21.42	36.50	52.30	93.53	118.00
σ_1(failure)Kg/sq. cm.	1.75	2.51	3.95	6.48	10.95	14.05

(b) CONSTANT VOLUME TESTS							
A_f sq.cm.	8.18	8.04	8.16	8.47	8.50	8.54	8.40
$(A_p - A_f)$ sq.cm.	1.82	1.96	1.84	1.53	1.50	1.46	1.60
σ_{3f} Kg/sq.cm.	2.40	2.40	2.50	3.32	4.00	7.00	9.00
$(\sigma_{3f} \times A_f)$ Kg.	4.37	4.70	4.60	5.08	6.00	10.22	14.40
Measured Downward Force Kg.	46.18	45.67	47.79	60.65	70.37	103.75	132.40
Net Downward Force Kg.	41.81	40.97	43.19	55.57	64.37	93.53	118.00
σ_1(failure)Kg/sq. cm.	5.11	5.10	5.29	6.56	7.57	10.95	14.05

later. Fig. 18 gives the curves obtained in each case, and it shows that the recorded values of upward force may be in error up to three per cent on the high side.

Computations for the vertical swelling pressure of five constant volume tests are given in Table IV. Values of vertical swelling pressure were found by dividing the net upward forces by the average area of the sample. Here there was a choice between using the average area at the end of consolidation (A_c), or the average area at failure (A_f) and the computations show that the maximum difference found between the two cases was 0.04 Kg/sq.cm.

Lambe (1960) has stated that a reduction can be expected in the term (R - A) of equation (2) when stresses acting on the sample are increased, but the magnitude of the change in (R - A) cannot be measured. This implies that an error of unknown magnitude remains uncompensated in the evaluation of the swelling pressure term, since the swelling pressure at failure was based on values measured before the shearing stresses were applied to the sample.

The overall view is that recorded values of vertical swelling pressure may be on the high side because of the error in calibration, and because of the unknown magnitude of the reduction in the interparticle electric forces. However the change allowed in length of the sample may have resulted in a lower value of swelling pressure, and

TABLE IV

CORRECTED VALUES OF VERTICAL SWELLING PRESSURE

σ_3(start)	Kg/sq.cm.	0.25	0.75	1.50	2.50	4.00
σ_3(end)	Kg/sq.cm.	2.40	2.40	2.50	3.32	4.00
A_c	sq.cm.	7.95	7.78	8.04	7.92	7.90
A_p	sq/cm.	10.00	10.00	10.00	10.00	10.00
(A_p-A_c)	sq.cm.	2.05	2.22	1.96	2.08	2.10
$(A_p-A_c)x\sigma_3$(end)	Kg.	4.92	5.33	4.90	6.91	8.40
A_f	sq.cm.	8.18	8.04	8.16	8.47	8.50
(A_p-A_f)	sq.cm.	1.82	1.96	1.84	1.53	1.50
$(A_p-A_f)x\sigma_3$(end)	Kg.	4.37	4.70	4.60	5.08	6.00
Measured Upward Force	Kg.	32.5	31.3	32.8	38.5	40.2
Net Upward Force (Based on A_c)	Kg.	27.58	25.97	27.90	31.59	31.8
Psv (Based on A_c)		3.47	3.34	3.47	3.99	4.02
Net Upward Force (Based on A_f)		28.1	26.6	28.2	33.4	34.2
Psv (Based on A_f)		3.43	3.31	3.46	3.95	4.02

the estimated correction for uplift on the piston may have made its contribution one way or the other, so that it is hard to estimate the percentage error in the values which were used.

2. Horizontal Swelling Pressure. During the time that swelling pressure in the horizontal direction was measured, plots were made on a log time basis, and these indicated that a maximum value was reached between one and four hours after water was made available to the sample. The longer time was needed for the samples which were started at a lower value of confining pressure. In the constant volume tests listed in Table IV, the samples were allowed access to water for seven or eight hours before the water was shut off for the night, so that this source should not cause any error in the value of horizontal swelling pressure obtained.

One factor which could have contributed to an error, and over which there was virtually no control was seepage around the piston. Exact measurement of the seepage volume was not possible, but it was estimated that with a good oil seal in the cell, seepage amounted to one and one half cubic centimetres within the eight hours that water was first made available to the sample. This volume change is about two and one half per cent of the sample volume.*

* This is based on an average sample volume of 57 ccs.

Increase in the water volume of the sample during the same period varied between two and three percent of the sample volume. Volume change in the sample leads to lower values of both vertical and horizontal swelling pressure, but it is quite difficult to make a quantitative assessment of the magnitude of this reduction.

c. Errors in the Measurement of Pore Pressure. In the constant volume tests pore pressure was measured from the top of the sample at the same time that water was made available to it at the bottom. Checks on these top measurements were made by measuring pore pressure from the bottom porous stone during the periods when the water supply to the sample was closed off.

The main source of error in pore pressure measurements is the trapping of air in the line between the sample and the null indicator. This source of error was reduced to a minimum by thoroughly flushing the lines with deaired water at the end of consolidation at 7.00 Kg/sq.cm. Deairing of the line to the top porous stone was a slow and tedious process, used to correct a condition which might not have existed if the face of the piston in contact with the porous stone was suitably grooved.

During the compression part of all tests, pore pressure was measured from the bottom of the sample, and here there was no problem in deairing the lines. All samples were strained at a con-

stant rate (1% per hour), and there was quite good agreement between the pore pressure obtained in corresponding tests.

A check was made to determine whether the chosen rate of strain was slow enough to allow full development of pore pressure in the sample, by running a test in the standard cell with a confining pressure of 4.00 Kg/sq.cms. and at a rate of strain of 0.25 per cent per hour. Pore pressure in this test was six per cent higher than the pore pressure obtained in a corresponding test carried out in the standard cell at the rate of strain used throughout (1% per hour), and five per cent higher than the pore pressure measured in a test carried out in the steel cell at the lower rate of strain. This discrepancy indicated that the rate of strain used in the tests was slow enough to allow a good measurement of pore pressure.

Pore pressure measurements were made without the application of back pressure to the sample. This step was neglected mainly because it would have been necessary to adopt this procedure in all tests for the sake of uniformity, and it was felt that applying back pressure to the constant volume tests would have the effect of introducing more unknowns in a situation that was already quite complex.

Back pressuring increases the degree of saturation of the sample, and is especially useful when tests are made on compacted samples, which generally have large volumes of air trapped in the

interparticle voids. All samples used were consolidated from the liquid limit and had degrees of saturation varying between ninety-five and ninety-nine per cent, so that it appears that the lack of back pressure introduced an error that may be neglected.

Results obtained in the Standard Tests - Standard Cell

Two important checks were made on the results of all tests. The first was based on the theory that a plot of moisture content versus the logarithm of compressive strength should fall on a line parallel to the virgin compression branch of the e-log p curve (Taylor 1948), and the second was based on the theory that for the same soil, the envelope for normally consolidated samples should be a straight line through the origin, while the envelope for over-consolidated samples should show some cohesion, and should cross the former envelope at the pre-consolidation pressure. (Terzaghi and Peck, 1948).

Two consolidation tests were run starting at the liquid limit, and loading to pressures of 16 Kg/sq.cm. Unfortunately, soil started oozing around the porous stone under the highest pressure, and the samples were not allowed to rebound. Fig. 7 shows the e-log p curve obtained in these tests. Fig. 8 is a plot of water content versus the logarithm of deviator stress at failure, made on the same sheet with a plot of the e-log p curves (dashed lines) of Fig. 7.

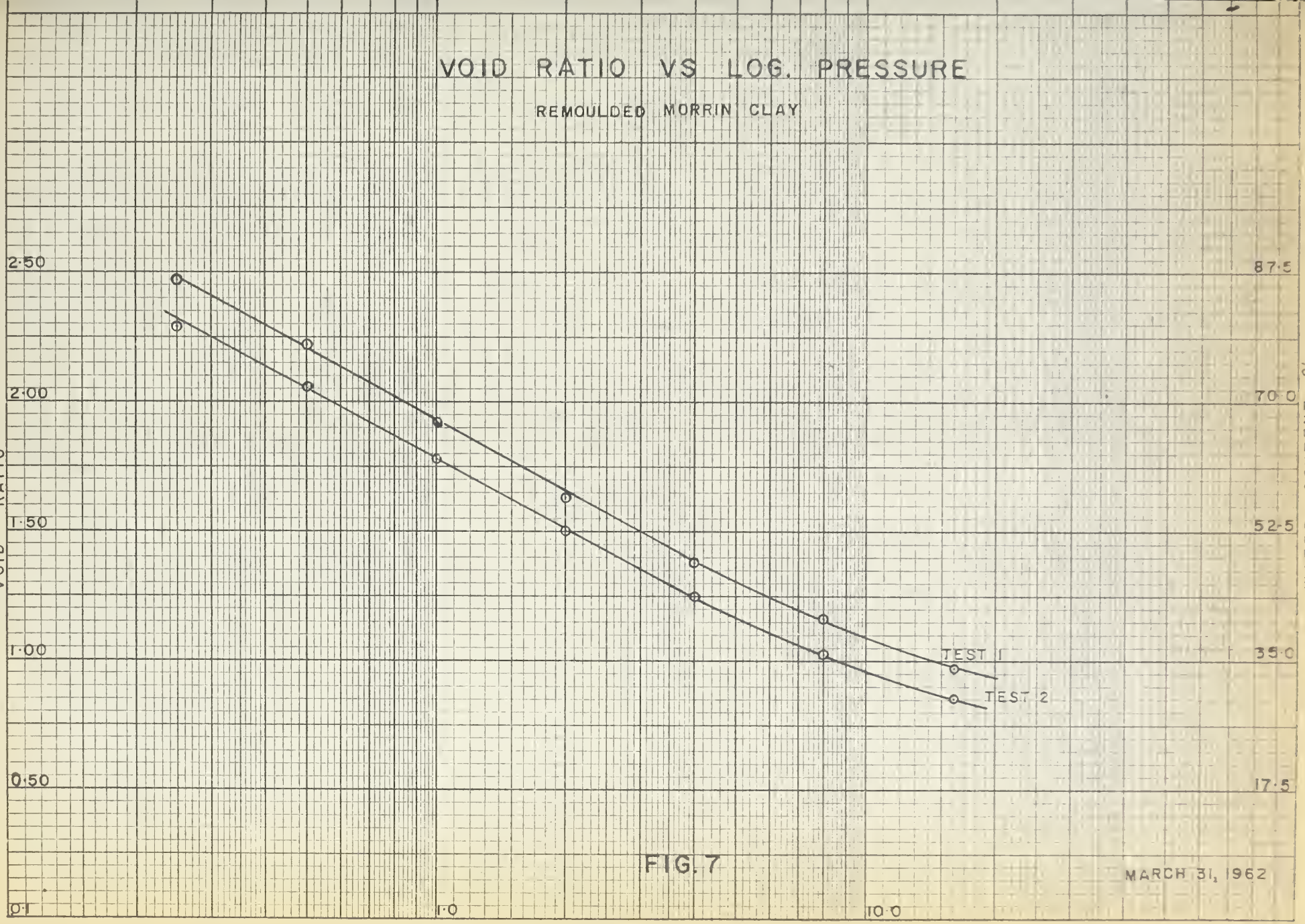

VOID RATIO VS LOG. PRESSURE
REMOULDED MORRIN CLAY
2·50
2·00
1·50
1·00
0·50
VOID RATIO
87·5
70·0
52·5
35·0
17·5
WATER CONTENT %
TEST 1
TEST 2
0·1
1·0
10·0
PRESSURE — kg/sq.cm.
FIG. 7
MARCH 31, 1962

FIG 7

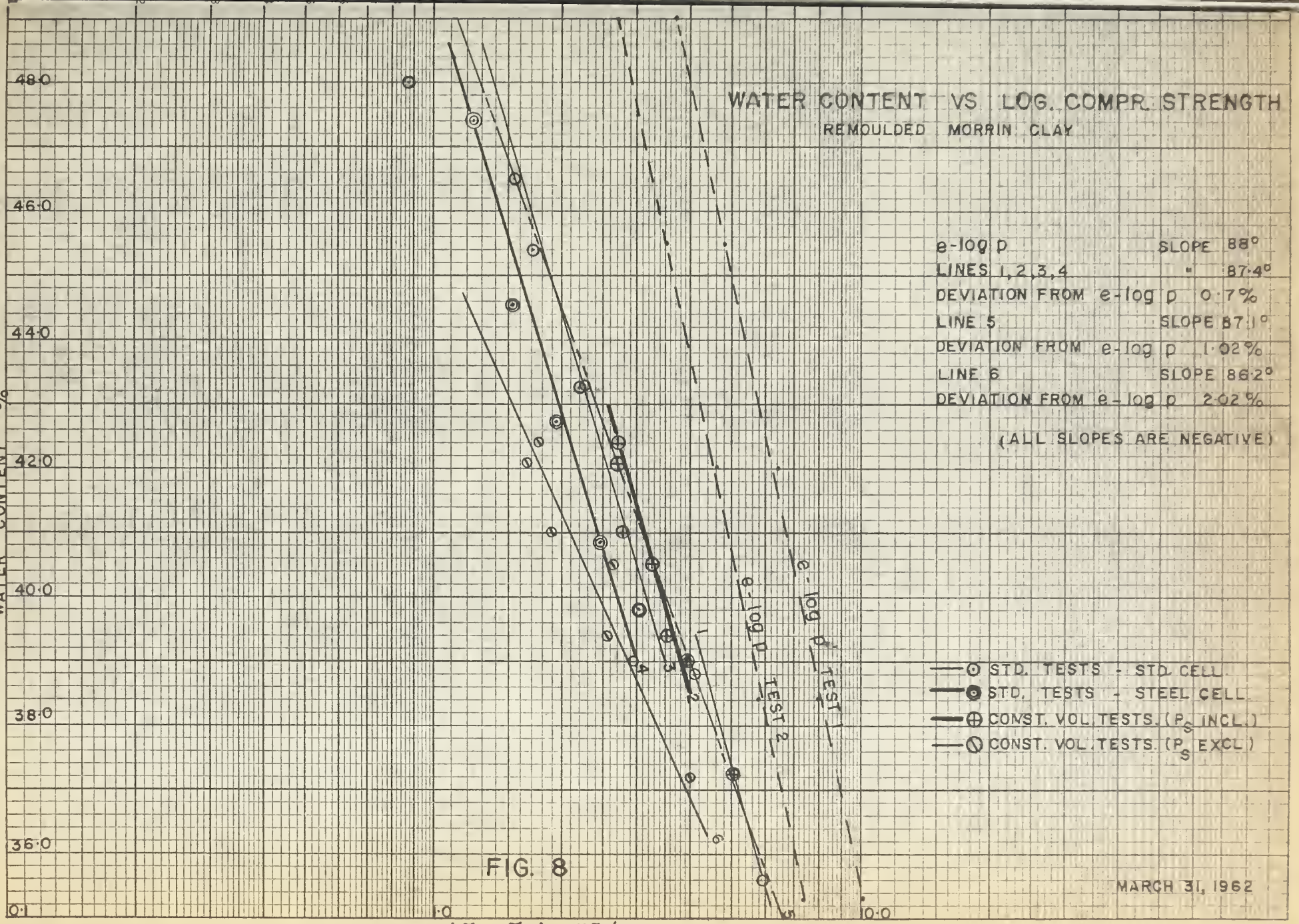


FIG. 8

MARCH 31, 1962

FIG 8

Line 3, Fig. 8, is the line joining points for overconsolidated samples tested in the standard cell, while line 1 joins points for normally consolidated samples in the same test series. Lines 1 and 3 are parrallel and slope at -87.4^{o}, 0.6^{o} less than the -88^{o} slope of the e-log p curves. Line 5 is the best fitting line for all points that have been plotted, and this deviates from the criterion by 0.9^{o}. This closeness of agreement helps to confirm the validity of the test results.

Fig. 8 can be used to adjust the diameters of the Mohr circles, if such an adjustment can aid in defining a better failure envelope. Adjustment is made by shifting the plotted points to the best fitting line, and using the value obtained as the diameter of the Mohr circle. In this case adjustment to the results of the standard tests will be made only for the test carried out at a confining pressure of 0.25 Kg/sq.cm. the deviator stress will be increased from 0.88 Kg/sq. cm. to 1.3 Kg/sq.cm. This is to help in better defining the cohesion intercept on the plot of Mohr circles.

Table V (a) gives a summary of the results of the standard tests, and the Mohr circles plotted from these results are shown in Fig. 9 (a) and Fig. 10 (a). The dashed circles in both of these figures are the adjusted circles mentioned in the preceding paragraph, and the indication is that the failure envelope curves downwards at

its lower extremity, a characteristic sometimes found in over-consolidated soil.

The effective value of the parameter ϕ for normally consolidated samples was 21^{o}, while the corresponding value for over-consolidated samples was 11.5^{o}. The parameter c had effective values of zero and 0.60 Kg/sq.cm. respectively. In the plot of total stress (Fig. 9(a)), the envelopes cross at the pre-consolidation load (7.00 Kg/ sq.cm.) and here again the test results agree with theory.

No pore pressure reaction tests were run on any of the samples, but values of the pore pressure coefficient $\bar{A}$ were calculated on the assumption that the coefficient B had a value of unity. All over-consolidated samples except those run at a confining pressure of 4.0 Kg/sq.cm. showed a rise in $\bar{A}$ at the start of the test, followed by a fall. This fall began at rates of strain varying between 0.2% and 1.00%*, but in no case did $\bar{A}$ attain negative values. This agrees with the observation reported by Hardy et al (1960) for tests on the over-consolidated clays of Northern Alberta. The sample tested at 4.00 Kg/sq.cm., and the normally consolidated samples showed no decrease in $\bar{A}$ during the test.

*Failure in these samples took place between 2.0% and 3.25% strain. For further information see table V.

The close agreement of the results obtained in the standard tests with theory and with the results reported by others, suggests that these results are suitable for the comparison which is to be made between them and the results obtained in the Constant Volume tests.

Results Obtained in the Standard Tests -- Steel Cell

These tests were run specifically to try and determine whether the design of the steel cell had any affect on the test results, and an attempt was made to duplicate the results obtained in the standard tests. Before such a comparison was made, a correction was applied to the observed values of total normal stress. (See Table III a). Table V b gives a summary of the results obtained in these tests, and Fig. 9c and Fig. 10c show the Mohr circles for total and effective stresses for these tests.

A comparison of these results with the results obtained in the standard tests show quite good agreement. Line 4 (Fig. 8) joins points representing values obtained in this test series, and although it lies below line 3 (Standard tests) it is parallel to it. No adjustments were made from Fig. 8 to the Mohr circles plotted for this test series. Fig. 9c shows that the envelope for the over-consolidated samples crosses the envelope for the normally consolidated samples at the pre-consolidation load, while Fig. 10c shows that the effective value of ϕ for these tests (10.5^{o}) is one degree less than the value found

TABLE V

SUMMARY OF RESULTS OF STANDARD TRIAXIAL TESTS

	(a) STANDARD CELL									(b) STEEL CELL					
σ_{3c}	0.25	0.50	1.00	2.00	2.00	4.00*	4.00	7.00	9.00	0.50	1.0	2.0	4.0	7.0	9.0
σ_{3f}	0.25	0.50	1.00	2.00	2.00	4.00	4.00	7.00	9.00	0.50	1.0	2.0	4.0	7.0	9.0
σ_1	1.13	2.05	2.71	4.27	4.20	7.47	7.01	11.04	14.82	1.75	2.51	3.95	6.48	10.95	14.05
P_{pf}	0.16	0.20	0.20	0.38	0.38	0.84	0.79	3.19	3.68	0.25	0.41	0.44	0.80	3.25	4.60
$(\sigma_1 - \sigma_3)$	0.88	1.55	1.71	2.27	2.20	3.47	3.01	4.04	5.82	1.25	1.51	1.95	2.48	3.95	5.05
σ_3'	0.09	0.30	0.80	1.62	1.62	3.16	3.21	3.81	5.32	0.25	0.59	1.56	3.20	3.75	4.40
σ_1'	0.97	1.85	2.51	3.89	3.82	6.63	6.22	7.85	11.14	1.50	2.10	3.51	5.68	7.70	9.45
% w	48.0	46.5	45.4	43.3	43.2	42.5	39.8	38.8	35.6	47.2	44.5	42.7	40.8	39.0	37.2
ef	1.52	1.37	1.35	1.26	1.31	1.14	1.19	1.17	1.05	1.40	1.31	1.27	1.14	1.11	1.08
% Strain	2.00	2.00	3.00	3.00	3.25	6.50	7.00	10.50	11.00	1.25	1.75	4.50	3.50	8.00	10.00
ϕ	56.1	45.9	31.1	24.3	24.1	20.4	18.7	20.3	20.6	44.6	36.1	23.8	20.5	24.2	25.7

*Rate of Strain 0.25%/Hr.
Other Tests 1.00%/Hr.
All Results taken at failure

MOHR CIRCLES + TOTAL STRESS

REMOULDED MORRIN CLAY

FIG 9

MARCH 31, 1962

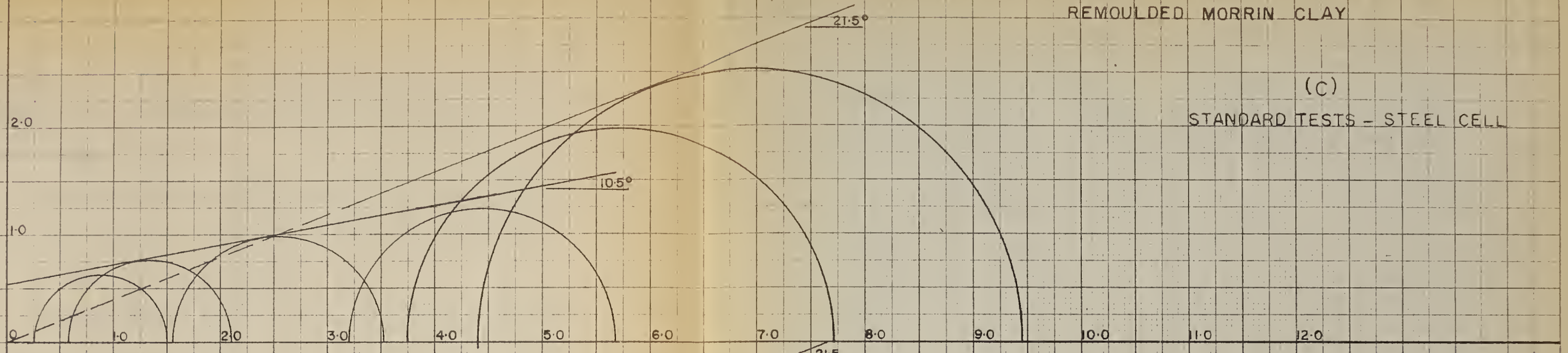
FIG 9

MOHR CIRCLES – EFFECTIVE STRESS

REMOULDED MORRIN CLAY

(c)

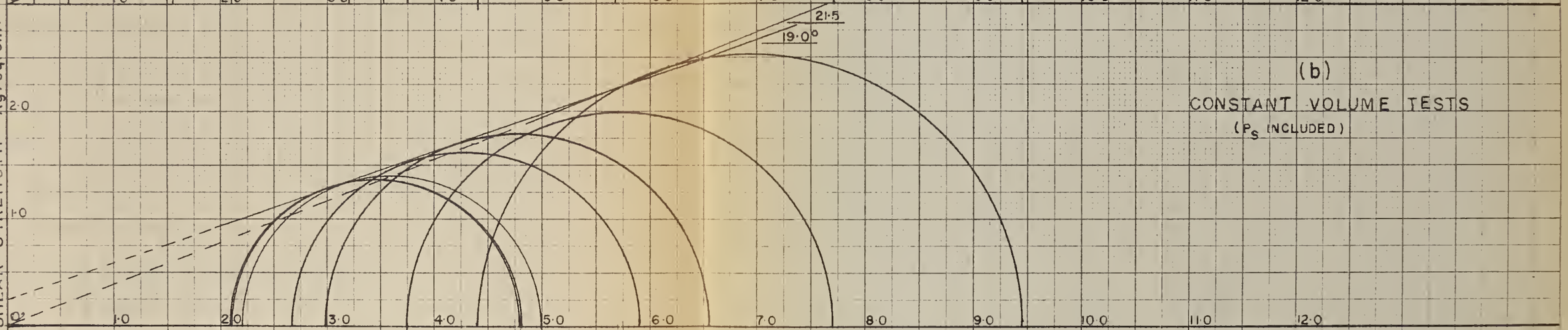
STANDARD TESTS – STEEL CELL



(b)

CONSTANT VOLUME TESTS

(P_s INCLUDED)



(a)

STANDARD TESTS – STANDARD CELL.

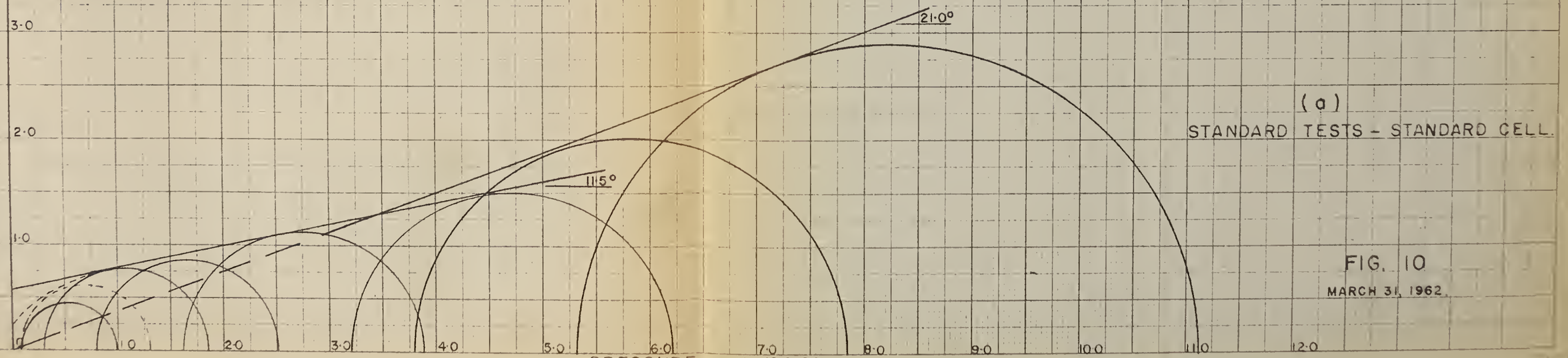


FIG. 10

MARCH 31, 1962.

FIG 10

in the standard tests(11.5°). The value of the effective cohesion is 0.55 Kg/sq.cm., 0.05 Kg/sq.cm. less than the value found in the standard tests.

The plot of total stress (Fig. 9c) defines a better failure envelope than the effective stress plot, but here again the difference in the value of ϕ is only one degree.

Values of $\bar{A}$ for these tests were computed on the assumption that the coefficient B was unity, and the results obtained followed the pattern described under Standard Tests.

The above comparison indicates that the design of the steel cell does not adversely affect the results of tests carried out in it, so that any differences which show up in the test results could be attributed to other causes.

Results obtained in the Constant Volume Tests

a. Swelling Pressure Included. In these tests swelling pressure was measured in both the horizontal and vertical directions, but the effective stress circles were drawn on the basis of Coulomb's equation (1). No reduction in stress was made for swelling pressure.

During the consolidation period of these tests pore pressure measurements were made from the top of the sample, while water was made available to it at the bottom, and while swelling pressure

was measured in both the horizontal and vertical directions. The values of pore pressure recorded ranged from 0.1 to -0.1 Kg/sq.cm.: the negative values were observed during the early stages of the consolidation period, and the positive values were observed during the final stages.

These values of pore pressure were measured at the same time that values of swelling pressure in excess of 2.0 Kg/sq.cm. were observed, and lend support to the theory that the interparticle, repulsive forces that are responsible for swelling are different from the pore pressure as measured by a piezometer, and cannot be detected by this means of measurement (Lambe 1960).

Table VI gives a summary of the results of these tests, and in the table it is seen that the samples which had initial values of confining pressure less than 2.4 Kg/sq.cm., required a stress of 2.4 to 2.5 Kg/sq.cm. to prevent volume increase, an indication that the horizontal swelling pressure of the soil was in the vicinity of 2.4 Kg/sq.cm.

One exception to this conclusion is the sample which had an initial confining pressure of 2.50 Kg/sq.cm. In this case the confining pressure had to be increased to 3.32 Kg/sq.cm. to prevent volume increase. This higher value of swelling pressure was quite unexpected, and may be due to a difference in the soil structure of

TABLE VI

SUMMARY OF RESULTS OF CONSTANT VOLUME TRIAXIAL TESTS

(SWELLING PRESSURE INCLUDED)

σ_{3i}	0.25	0.75	1.50	2.50	4.00	7.00	9.00
σ_{3f}	2.40	2.40	2.50	3.32	4.00	7.00	9.00
σ_1	5.11	5.10	5.29	6.56	7.57	10.95	14.05
P_{pf}	0.30	0.31	0.30	0.66	1.02	3.25	4.60
$(\sigma_1 - \sigma_3)$	2.71	2.70	2.79	3.24	3.57	3.95	5.05
σ_3'	2.10	2.09	2.20	2.66	2.98	3.75	4.40
σ_1'	4.81	4.79	4.99	5.90	6.55	7.70	9.45
% w	42.1	42.4	41.0	40.5	39.4	39.0	37.2
ef	1.25	1.22	1.21	1.20	1.11	1.11	1.08
% Strain	2.50	3.00	1.25	6.00	6.50	8.00	10.00
ϕ	25.9	26.3	25.7	24.7	24.7	24.2	25.7

Rate of Strain 1%/Hr.
All Results taken at failure

the sample. The procedure used for preparing and testing all samples was the same, so that there is no apparent reason for this discrepancy.

Values of vertical swelling pressure found in the first three tests of Table VI had an average of 3.4 Kg/sq.cm., and this is considered as the vertical swelling pressure of the soil.

The samples which had initial confining pressures greater than 2.50 Kg/sq.cm. needed no pressure increase to maintain constant volume, so that their horizontal swelling pressure could not be measured. The horizontal swelling pressure of these samples was assumed to be 2.4 Kg/sq.cm., the average value found in the first three tests.

The magnitude of the vertical swelling pressure of the soil (3.4 Kg/sq.cm.) is much greater than the values of swelling pressure that have been measured in consolidation tests carried out on the undisturbed soil. Fig. 11 is the pressure versus void ratio curve for a constant pressure consolidation test carried out on an undisturbed sample of the soil used in this investigation. In this figure it is seen that the swelling pressure is 1.45 Kg/sq.cm., a value much less than the values found in the triaxial cell. The difference in these values of swelling pressure may be due to the difference between the concentration of the salts in the double layers of the remoulded and undisturbed samples. Drying a sample removes the water, but the salts remain in the soil. When the sample is remoulded with distilled water, the concentration of the salts in the double layer may be dif-

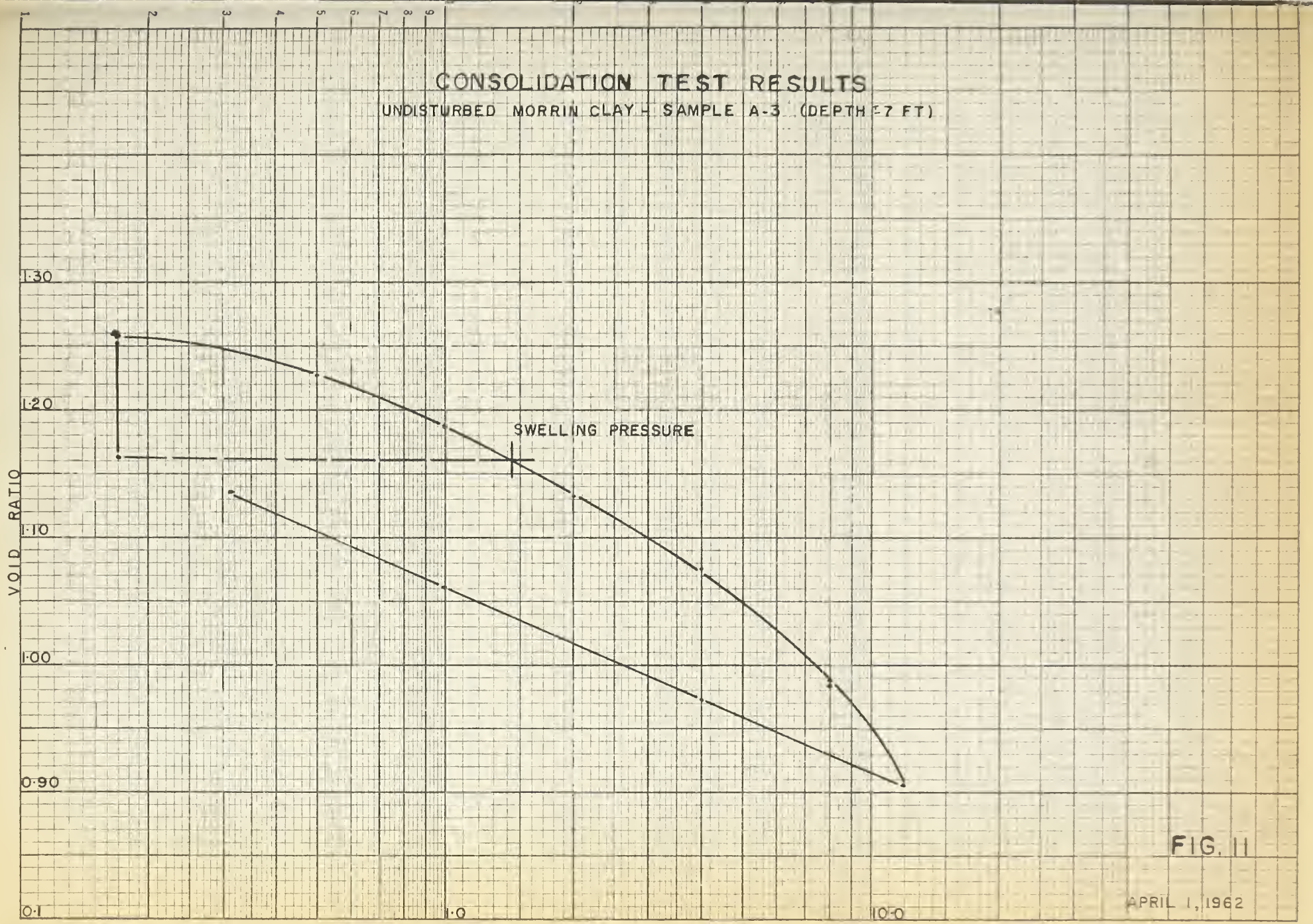

CONSOLIDATION TEST RESULTS
UNDISTURBED MORRIN CLAY - SAMPLE A-3 (DEPTH 7 FT)
SWELLING PRESSURE
VOID RATIO
1·30
1·20
1·10
1·00
0·90
0·1
1·0
10·0
PRESSURE — Kg/sq. cm.
APRIL 1, 1962

FIG. 11

ferent from what it was formerly, and this could possibly be the reason for the observed difference in swelling pressure.

A similar situation may be duplicated in the field, when moisture is lost from the top layer of the soil. In this zone there is a change in the salt concentration of the double layers, so that when rain water runs through cracks and fissures, it is adsorbed by the soil before the salt content of the water is appreciably increased, and this could lead to greater values of swelling pressure than those found by laboratory tests.

Another observation that can be made is that the effective angle of internal friction found from Standard tests on over-consolidated remoulded samples is 11.5 degrees, a value which very closely approaches the steepness of the long term slopes in Northern Alberta. (Average values 9 - 12 degrees). Of course, the angles are not directly comparable, because the laboratory tests show an appreciable value of cohesion. Even though the tests were carried out on a soil which differs from the soils found in Northern Alberta, the implication is that over-consolidating causes a decrease in the angle of internal friction, and on a long term basis, an analysis to determine safe angles of slope may be obtained by determining strength characteristics on remoulded over-consolidated soil samples. The over-consolidated soils of Northern Alberta are known to have a top layer which is often cracked and fissured, and this top layer may be considered to be in a

remoulded state, so that the use of this approach for finding safe slope angles should be further investigated.

In the Constant Volume tests, the tendency for the samples to require a minimum value of confining pressure to prevent volume increase, had the effect of giving Mohr circles which plotted from the value of swelling pressure (Fig. 9b). In this plot two strength envelopes are shown lying parallel to each other. If the samples were truly held at constant volume there would be very little or no variation in their water content, and they would all be expected to have the same strength, and so give a horizontal failure envelope. Due to imperfections in the apparatus, volume increases of approximately 5 per cent took place in the samples, and this led to increases in the water content, and lower values of strength. This would account for the Mohr circles being shifted to the left, but the reason for the parallelism of the failure envelopes is not known. It may be merely coincidence.

A plot of water content versus the logarithm of compressive strength (Fig. 8 line 2) falls parallel to the line representing the results of the Standard tests, and satisfies this criterion.

The plot of Mohr circles of effective stress is shown in Fig. 10b. Two envelopes are drawn, but the effective value of $\emptyset$ may be considered constant at 21.5 degrees, since the difference shown is small, and

may be explained by the fact that the samples did undergo a small volume increase.

A comparison between the strength envelopes for the Standard tests and the Constant Volume tests show that on the basis of total stress, as well as effective stress, both test Series give virtually the same strength envelope for values of confining pressure greater than the horizontal swelling pressure of the soil. This result is to be expected since at these values of confining pressure, no volume change can take place in the samples in either test, so that the test conditions are identical in both cases. This means that the swelling pressure is present in both types of test. One important point to be noticed here is the lack of information provided by the Constant Volume tests for samples tested at pressures less than the horizontal swelling pressure. In this case backward extrapolation of the strength envelope (Fig. 12) would lead to lower values of strength than may be obtained in the Standard test. However it is not known whether the Constant Volume tests would follow the extrapolated line, or the envelope defined by the Standard tests, or some other line.

One additional circle could have been obtained in the Constant Volume test series, by holding the normal stress constant after the horizontal and vertical swelling pressures had reached equilibrium values, and then causing failure to occur by a reduction of the lateral stress. This procedure would have given an indication as to the position of the strength envelope in this region.

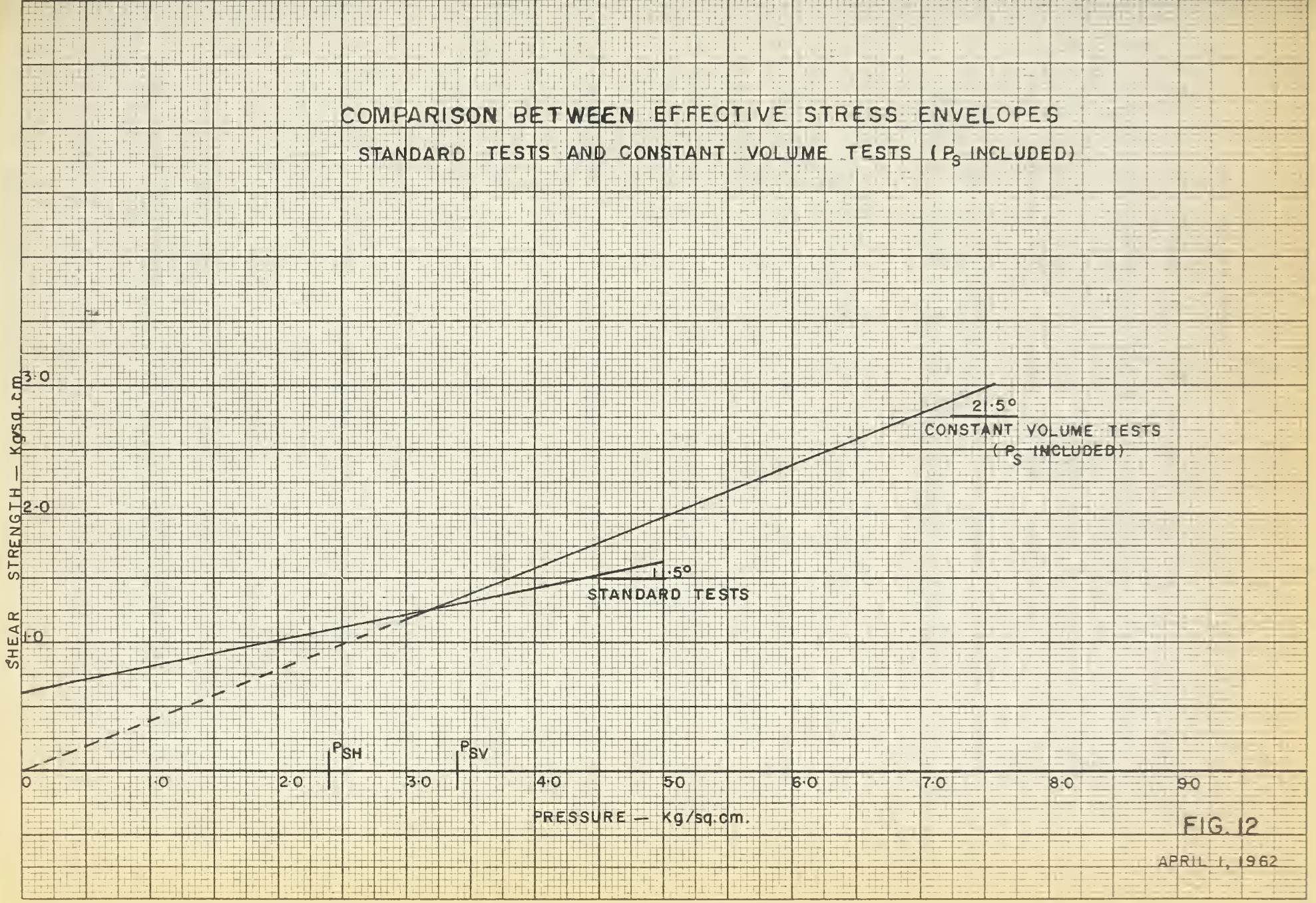


FIG. 12

APRIL 1, 1962

FIG. 12

Various investigators (Bishop and Bjerrum 1960, Skempton and De Lory 1957) have reported that results obtained from Standard tests usually give an over estimate of the strength of over-consolidated clays, and so a knowledge of the position of the Constant Volume strength envelope may reveal whether Constant Volume tests are better suited for estimating strength values of the soil at values of confining pressure less than the horizontal swelling pressure of the soil.

It may be significant that the slides mentioned in Chapter I took place at a relatively shallow depth, or in a zone in which the overburden pressure could possibly be less than the swelling pressure of the soil. Since the test results yield no information on changes in strength that may possibly take place in this zone, it is strongly recommended that an investigation be undertaken to attempt to determine the position of the failure envelope in this area, even if only one failure circle can be obtained because of the limiting conditions of the test procedure.

Swelling pressure may well act in a manner to reduce the shear strength of the soil, but at present there is no known method of measuring this effect. All tests which are run at values of lateral stress higher than the swelling pressure of the soil will automatically include the swelling pressure in the results. One avenue of approach that would be worth investigating would be an attempt to establish a relationship

between the swelling pressure of the soil, and the effective angle of internal friction. In the literature the effective angle of internal friction for cohesive soils has been reported to range between 20^{o} and 32^{o} (Bishop and Bjerum 1960), and it may be that the high values of ϕ are associated with soils having a low swelling pressure, while the lower values of ϕ are associated with soils having a high swelling pressure.

Of course the so called swelling pressure is not a constant for a soil, but depends upon the previous stress history; the nature of the pore fluid, the orientation of the soil grains, and perhaps other factors as well.

CHAPTER VIII

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions have been reached in this investigation.

1. A literature search reveals that swelling in soils may best be explained by the Osmotic Pressure theory of swelling, or by the concept of capillary tension release, (Locked Pressure theory) or by a combination of the two.
2. The test results show that swelling pressure is distinct from pore pressure as measured by a piezometer, and cannot be detected by this means of measurement.
3. The test results show that swelling pressure is greater in the vertical direction than in the horizontal direction.
4. No positive conclusions can be drawn as to whether swelling pressure acts as an effective stress or as a neutral stress.
5. The test results show that Constant Volume tests in which pore pressure and swelling pressure are measured give Mohr circles that fit the strength envelopes obtained in the Standard tests, provided that the confining pressure is greater than the swelling pressure of the soil. This is to be expected since there does not appear to be any way of conducting a test at confining pressures greater than the swelling pressure without automatically recording the effects of the swelling pressure.

The following recommendations are made for future research.

1. An investigation to determine the position of the failure envelope for samples tested at confining pressures less than the horizontal swelling pressure, by both Constant Volume and Standard tests, and for several soils with varying swelling pressures.

2. An investigation to determine whether there is a correlation between the swelling pressure of the soil and the effective angle of internal friction.

3. An investigation to determine the changes in swelling pressure and shear strength which would be brought about by a change in the electrolyte concentration of the water with which the samples are remoulded.

4. Improvement of the equipment by providing closer machine tolerances between the piston, and the brass bushing, so that seepage losses around the piston may be substantially reduced. Careful lapping of the piston should be adequate.

BIBLIOGRAPHY

BIBLIOGRAPHY

1. Alpan, I. (1959) A Study of the Principle of Effective Stress in Partly Saturated Soils. Ph. D. thesis. Imperial College of Science and Technology, University of London.

2. Balmer, G. (1952). A General Analytic Solution for Mohr's Envelope. Proceedings ASCE Vol. 78 pp. 1260-1270.

3. Baracos, A (1958). Shrinkage and Swelling of Clays in Western Canada. Proceedings of the 12th Canadian Soil Mechanics Conference. Technical Memorandum No. 59.

4. Baver, L. D. (1956). Soil Physics. 3rd Edition, Wiley and Sons Inc. New York.

5. Bishop, A. W. (1954). The Use of the Pore Pressure Coefficients in Practice. Geotechnique Vol. 4 pp. 148-152.

6. Bishop, A. W. (1960). The Measurement of Pore Pressure in the Triaxial Test. Paper presented at the conference on Pore Pressure and Suction in Soils. Butterworth. London.

7. Bishop, A. W. and Bjerrum, L. (1960). The Relevance of the Triaxial Test to the Solution of Stability Problems. Norwegian Geotechnical Institute. Publication No. 34.

8. Bishop, A. W. and Henkel, D. J. (1957). The Measurement of Soil Properties in the Triaxial Test. Edward Arnold, London.

9. Bjerrum L. (1954). Theoretical and Experimental Investigation on the Shear Strength of Soils. Norwegian Geotechnical Institute, Publication No. 5.

10. Bolt, G. H. (1956). Physico Chemical Analysis of the Compressibility of Pure Clays. Geotechnique Vol. 6 pp. 86-93.

11. Bozozuk, M. (1958). Volume Changes Measured in Leda Clay. Proceedings of the 12th Canadian Soil Mechanics Conference. Technical Memorandum No. 59.

12. Casagrande, A. (1932). The Structure of Clay in Foundation Engineering. Boston Society of Civil Engineers -- Contributions to Soil Mechanics 1925 - 1940.

13. Casagrande, A. and Rivard, P. J. (1959). Strength of highly Plastic Clay. Norwegian Geotechnical Institute. Publication No. 31. Also Harvard Soil Mechanics Series No. 60.

14. Casagrande A. and Wilson, S. D. (1951). Effect of Rate of Loading on the Strength of Clays and Shales at Constant Water Content. Geotechnique Vol. 2 pp. 251-263.

15. De Bruijn, C. M. A. (1961). Swelling Characteristics of a Transported Soil Profile at Leeuhof, Vereeniging (Transvaal). Proceedings of the 5th International Conference on Soil Mechanics and Foundation Engineering. Vol. 1 pp. 43-48.

16. Falconer, J. G. and Mattson, S. (1933). The Laws of Soil Colloidal Behaviour XII. Osmotic Imbibition. Soil Science Vol. 36 pp. 317-327.

17. Finn, W. D. and Strom, B. (1958). Nature and Magnitude of Swelling Pressure. Proceedings Highway Research Board Vol. 37 pp. 493-505.

18. Grim, R. E. (1940). The Clay Minerals in Soils and Their Significance. Purdue Conference on Soil Mechanics pp. 216-222.

19. Grim, R. E. (1958). Organization of Water on Clay Mineral Surfaces, and its Implications for the Properties of Clay Water Systems. Highway Research Board. Special Report 40.

20. Grim, R. E. (1959). Physico-Chemical Properties of Soils -- Clay Minerals. Journal of Soil Mechanics and Foundations Division -- Proceedings ASCE Vol. 85 No. SM 2. Part I pp. 1 - 16.

21. Hardy, R. M. (1957). Engineering Problems Involving Preconsolidated Clay Shales. Transactions of the Engineering Institute of Canada No. I.

22. Hardy, R. M. Brooker, E. W., Curtis, W. E. (1961) Land Slides in Over-Consolidated Clays. Paper presented to the 75th annual meeting of the E.I.C.

23. Hemwall, J. B. and Low, P. F. (1956). The Hydrostatic Repulsive Force in Clay Swelling. Soil Science Vol. 82 pp. 135-145.

24. Henkel, D. J. (1957). Investigation of Two Long Term Failures in London Clay Slopes at Wood Green and Northolt. Proceedings of the 4th International Conference on Soil Mechanics and Foundation Engineering Vol. 11 pp. 315-319.

25. Henkel, D. J. (1960). The Shear Strength of Saturated Remoulded Clays. Paper presented at the ASCE research conference on Shear Strength of Cohesive Soils. Boulder, Colorado.

26. Holtz, W. G. (1959). Expansive Clays -- Properties and Problems. Quarterly of the Colorado School of Mines. Vol. 54 No. 4 pp. 91-125.

27. Holtz, W. G. and Gibbs, H. J. (1956). Engineering Properties of Expansive Clays. Transactions ASCE Vol. 121 paper 2814 pp. 641-663.

28. Katz, J. R. (1933). The Laws of Swelling. Transactions of the Faraday Society. Vol. 29 pp. 279-300.

29. Ladd, C. C. (1960). Mechanics of Swelling by Compacted Clays. Highway Research Board. Bulletin 245. Washington, D. C. pp. 10-26.

30. Lambe, T. W. (1951). Soil Testing for Engineers. Wiley and Sons Inc. New York.

31. Lambe, T. W. (1953). The Structure of Inorganic Soil. Proceedings ASCE Vol. 79. Separate No. 315.

32. Lambe, T. W. (1958a). The Structure of Compacted Clay. Journal of the Soil Mechanics and Foundations Division. Proceedings ASCE Vol. 84 SM 2. Paper 1654.

33. Lambe, T. W. (1958b). The Engineering Behaviour of Compacted Clays. Journal of the Soil Mechanics and Foundation Division. Proceedings ASCE Vol. 84 SM 2. Paper 1655.

34. Lambe, T. W. (1960) A Mechanistic Picture of the Shear Strength in Clay. Paper presented at the ASCE research conference on Shear Strength of Cohesive Soils. Boulder, Colorado.

35. Lambe, T. W. and Whitman, R. V. (1959). The Role of Effective Stress in the Behaviour of Expansive Soils. Quarterly of the Colorado School of Mines. Vol. 54 No. 4 pp. 35-60.

36. Lowe, J. and Karafiath, L. (1960). Effect of Anisotropic Consolidation on the Undrained Shear Strength of Compacted Clays. Paper presented at the ASCE research Conference on Shear Strength of Cohesive Soils. Boulder, Colorado.

37. Means. R. E. (1959) Building on Expansive Clay. Quarterly of the Colorado School of Mines. Vol. 54. No. 4 pp. 5-31.

38. Michaels, A. S. (1959). Discussion on--Physico-Chemical Properties of Soils--Soil Water Systems. Journal of the Soil Mechanics and Foundation Division. Proceedings ASCE Vol. 85. SM 2. pp 91-102.

39. Michaels, A. S. and Lin, C. S. (1954). The Permeability of Kaolinite. Industrial and Engineering Chemistry. Vol. 46. No. 6. pp. 1239-1246.

40. Peck, R. B., Hanson, W. E., Thornburn, L. H. (1959) Foundation Engineering. Wiley and Sons Inc. New York.

41. Peterson, R. (1958) Rebound in Bearpaw Shale. Bulletin of the Geological Society of America. Vol. 69. pp. 1113-1123.

42. Peterson, R., Rivard, P. J., Jasper, J. L., Iverson, N. L. (1960). Limitations of Laboratory Shear Strength in Evaluating Stability of Highly Plastic Clays. Paper presented at the ASCE research conference on Shear Strength of Cohesive Soils. Boulder, Colorado.

43. Puri, A. N. (1939). Physical Characteristics of Soil. Soil Science Vol. 48. pp. 505-520.

44. Rosenquist, I. Th. (1959). Physico-Chemical Properties of Soils Soil Water Systems. Journal of the Soil Mechanics and Foundations Division. Proceedings ASCE Vol. 85 No. SM 2. Part I. pp 31-52.

45. Schmertmann, J. H., and Osterberg, J. O. (1960). An Experimental Study of the Development of Cohesion and Friction with Axial Strain in Saturated Cohesive Soils. A Northwestern University research paper.

46. Sebastyan, G. Y. (1958). Discussion on -- The Mechanism of Volume Change in Clay. Proceedings of the 12th Canadian Soil Mechanics Conference. Technical Memorandum No. 59.

47. Sideri, D. T. (1936). Soil Swelling I. The Swelling of Soils in Water considered in connection with the Problem of Soil Structure. Soil Science, Vol. 41 pp. 135-151.

48. Skempton, A. W. (1954). The Pore Pressure Coefficients A and B. Geotechnique Vol. 4 pp. 143-147.

49. Skempton, A. W. (1960). Effective Stress in Soils, Concrete and Rocks. Paper presented at the conference on Pore Pressure and Suction in Soils. London.

50. Skempton, A. W., and De Lory, F. A. (1957). Stability of Natural Slopes in London Clay. Proceedings 4th International Conference on Soil Mechanics and Foundations, Vol. II pp. 378-381.

51. Taylor, A. W. (1959). Physico-Chemical Properties of Soils--Ion Exchange Phenomena. Journal of the Soil Mechanics and Foundations Division. Proceedings ASCE Vol. 85 No. SM 2. Part I pp. 19-29

52. Taylor, D. W. (1948). Fundamentals of Soil Mechanics. New York. John Wiley and Sons Inc.

53. Terzaghi, K. (1943). Theoretical Soil Mechanics. John Wiley and Sons Inc. New York.

54. Terzaghi, K., and Peck, R. B. (1948). Soil Mechanics in Engineering Practice. John Wiley and Sons Inc. New York.

55. Warkentin, B. P. (1958). The Mechanism of Volume Change in Clays. Proceedings of the 12th Canadian Soil Mechanics conference. Technical Memorandum No. 59.

56. Whitman, R. V. (1960). Some Considerations and Data Regarding the Shear Strength of Clay. Paper presented at the ASCE research conference on Shear Strength of Cohesive Soils. Boulder, Colorado.

57. Wyatt, F. A., and Newton, J. D. (1943). Soil Survey of Rosebud and Banff Sheets. Bulletin No. 40. Distributed by Department of Extension, University of Alberta, Edmonton, Alberta.

58. Yoder, E. J. (1959). Principles of Pavement Design. Wiley and Sons Inc. New York.

APPENDICES

APPENDIX I

DESIGN OF THE STEEL TRIAXIAL CELL

Design of the Steel Triaxial Cell

Length of Cell	= 5"
Diameter of Cell	= 2.5"
Thickness of cell walls	= 1/4"
Cross sectional area of cell	= 4.91 in.^{2}
Volume of cell	24.55 cu. in. 402.3 ccs.
Diameter of Piston	= 1.4"
Cross sectional area of piston	= 10 cm^2
Volume of cell occupied by piston & sample	= 127 ccs.
Volume of fluid in cell	275.30 ccs.
Diameter of specimen	= 1.4"
Cross sectional area of specimen	= 10 cm^2
Length of specimen	= 8 cm
Volume of specimen	= 80 ccs.
Ratio $\frac{\text{Length of specimen}}{\text{Diameter of specimen}}$	= 2.25
Ratio $\frac{\text{Volume of specimen}}{\text{Volume of cell}}$	= 0.199
Ratio $\frac{\text{Volume of specimen}}{\text{Volume of fluid}}$	= 0.291

Since the cell is held to the base by short bolts which do not pass through the top cap, there will be a tendency for the cell to elongate when the pressure is applied inside. A calculation will be made to determine the magnitude of this elongation.

Assume a maximum pressure within the cell of 10 Kg/cm^2

According to Hooke's Law $\delta = \frac{PL}{AE}$

where P = Total Force acting on the cylinder = pA = 10A

L = Length of the cell = 5"

A = Cross sectional area of the cell

E = Young's modulus for steel = 30×10^6 lb/in^2

δ = Elongation of the cell.

Since 1 $Kg/cm^2 \approx$ 1 ton/ft^2 we may write

$$\delta = \frac{10 \times 2000 \text{ lb}}{144 \text{ in}^2} \times \frac{A}{A} \times \frac{5 \text{ in}}{30 \times 10^6} \quad \frac{\text{in}^2}{\text{lb}}$$

$$= \frac{10 \times 10^4}{4.32 \times 10^9}$$

$$\delta = 2.32 \times 10^{-5} \text{ ins.}$$

Thus the elongation resulting from the tensile stress due to a pressure of 10 Kg/cm^2 inside the cell is negligibly small. Since it is not planned to use pressures in excess of 10.00 Kg/cm^2 in the tests, elongation of the cell will not be considered when calculating volume changes due to pressure inside the cell.

Calculation of Volume Changes

When a closed cylinder of inside diameter "D", height "h" and wall thickness "t" is subjected to a fluid pressure "p" p.s.i., the hoop stress set up in the walls of the cylinder is

$$P = p \frac{D}{2} \quad h$$

The deformation resulting from this stress may be calculated by Hooke's Law $\delta = \frac{PL}{AE}$

For the cylinder under consideration the change δ would be a change in circumference, and the values of P, L, A and E would be the same as given on page 95

When pressure is applied to the cell the original value of the circumference of cell (π D) will increase by an amount ($\delta \pi$ D), the magnitude of this increase depending on the pressure "p" inside the cell. Combining the above expressions we may write:-

$$\delta \pi D = \frac{(\frac{1}{2} p D h)\, \pi D}{t\ h\ E}$$

from which we get that

$$\delta D = \frac{pD^2}{2Et}$$

By use of this expression, the change in diameter of the cell corresponding to any value of pressure "p" can be found, and from this the change in volume can be calculated.

Allowable stress in the cell.

The maximum stress in the walls of the cell will be

$$s_{max} = \frac{P}{A} = \frac{pDh}{2ht}$$

$$s_{max} = \frac{pD}{2t}$$

Assuming that the allowable load in tension for steel is 20,000 p.s.i. then the allowable stress inside the cell is

$$P_{(all.)} = \frac{20,000\ (2t)}{D}\ lbs/in^2$$

Since $t = 1/4''$ and $D = 2.5''$

$$P_{(all.)} = \frac{20,000}{5}\ lb/in^2$$

$$= 4,000\ lb/in^2$$

This means that the walls of the cell will not fail under the loads that will be used during the test.

Sample Calculation

For $t = 1/4''$

$D = 2.5''$

$h = 5''$

$p = 1\ ton/ft^2$

$E = 30 \times 10^6\ lb/in^2$

$$\delta D = \frac{pD^2}{2Et}$$

$$= \frac{2000 \times (2.5)^2 \times 4}{2 \times 144 \times 30 \times 10^6}$$

$$= \frac{12.50 \times 10^3}{8.64 \times 10^9}$$

$$= 1.447 \times 10^{-6}\ ''$$

This calculation shows that if the cell is subjected to a pressure of 1 ton per sq.ft. then the diameter will increase 1.447×10^{-6} inches. If the pressure in the cell was 10 tons/ft^2 then the increase in diameter will be 1.447×10^{-5} inches. Within the pressure range of 1 to 10 tons per square foot we may safely conclude that the change in diameter of the cell will be so small that the corresponding volume change may be regarded as negligible.

Table VIII gives the results of computations made to determine the relationship between volume change and pressure for lucite and steel triaxial cells of varying length, diameter, and wall thickness, and these results are plotted in Fig. 14. A plot of volume change versus temperature change for lucite and steel cells is given in Fig. 15.

TABLE VIII

RELATIONSHIP BETWEEN VOLUME CHANGE AND PRESSURE FOR VARIOUS TRIAXIAL CELLS

CELL "d"- LUCITE THICKNESS .25" LENGTH 11.6" DIAMETER 4.75"

PRESSURE Kg/sq cm.	Δ D inches	AREA sq. inches	VOLUME ccs	Δ V ccs
0	-	17.712	3369.53	-
1	13.9×10^{-4}	17.722	3371.51	1.98
2	27.8×10^{-4}	17.732	3373.32	3.79
3	41.7×10^{-4}	17.743	3375.45	5.92
4	55.6×10^{-4}	17.754	3377.58	8.05
5	69.5×10^{-4}	17.764	3379.38	9.85
6	83.4×10^{-4}	17.773	3381.19	11.66
CELL "e"- LUCITE THICKNESS .375" LENGTH 7.5" DIAMETER 3.0"				
0	-	7.065	868.995	-
1	4.0×10^{-4}	7.0669	869.229	.234
2	8.0×10^{-4}	7.0688	869.462	.467
3	12.0×10^{-4}	7.0707	869.696	.701
4	16.0×10^{-4}	7.0725	869.918	.923
5	20.0×10^{-4}	7.0744	870.151	1.156
6	24.0×10^{-4}	7.0763	870.385	1.390
CELL "f" LUCITE THICKNESS .5" LENGTH 7.5" DIAMETER 3.0"				
0	-	7.065	868.995	-
1	3.0×10^{-4}	7.0664	869.167	.172
2	6.0×10^{-4}	7.0678	869.339	.344
3	9.0×10^{-4}	7.0692	869.512	.517
4	12.0×10^{-4}	7.0707	869.696	.701
5	15.0×10^{-4}	7.0721	869.868	.873
6	18.0×10^{-4}	7.0735	870.041	1.046

E - STEEL 30×10^6 LUCITE 4.2×10^5 lbs/sq.in.

TABLE VIII (CONTINUED)

RELATIONSHIP BETWEEN VOLUME CHANGE AND PRESSURE FOR VARIOUS TRIAXIAL CELLS

CELL "a"-STEEL. THICKNESS ·25" LENGTH 6·75" DIAMETER 3·5"

PRESSURE kg/sq.cm.	ΔD inches	AREA sq. inches	VOLUME (V) ccs	−V ccs
0	—	9·61625	1063·87	—
1	11 × 10⁻⁶	9·61631	1063·881	·006
2	22 × 10⁻⁶	9·61637	1063·887	·012
3	33 × 10⁻⁶	9·61643	1063·894	·019
4	44 × 10⁻⁶	9·61649	1063·901	·026
5	55 × 10⁻⁶	9·61655	1063·908	·033
6	66 × 10⁻⁶	9·61661	1063·914	·039
CELL "b"- STEEL. THICKNESS ·25" LENGTH 6·75" DIAMETER 4·0"				
0	—	12·560	1389·55	—
1	15 × 10⁻⁶	12·5601	1389·560	·012
2	30 × 10⁻⁶	12·5602	1389·573	·023
3	45 × 10⁻⁶	12·5603	1389·584	·034
4	60 × 10⁻⁶	12·5604	1389·595	·045
5	75 × 10⁻⁶	12·5605	1389·606	·056
6	90 × 10⁻⁶	12·5606	1389·619	·069
CELL "c" LUCITE. THICKNESS ·34" LENGTH 7·7" DIAMETER 4·91"				
0	—	18·925	2389·81	—
1	11·0 × 10⁻⁴	18·933	2390·86	1·05
2	21·9 × 10⁻⁴	18·942	2391·99	2·18
3	32·9 × 10⁻⁴	18·951	2393·14	3·33
4	43·8 × 10⁻⁴	18·959	2394·14	4·33
5	54·7 × 10⁻⁴	18·967	2395·15	5·34
6	65·7 × 10⁻⁴	18·976	2396·29	6·48

E-STEEL 30×10^6 LUCITE $4{\cdot}2 \times 10^5$ lbs/sq.in.

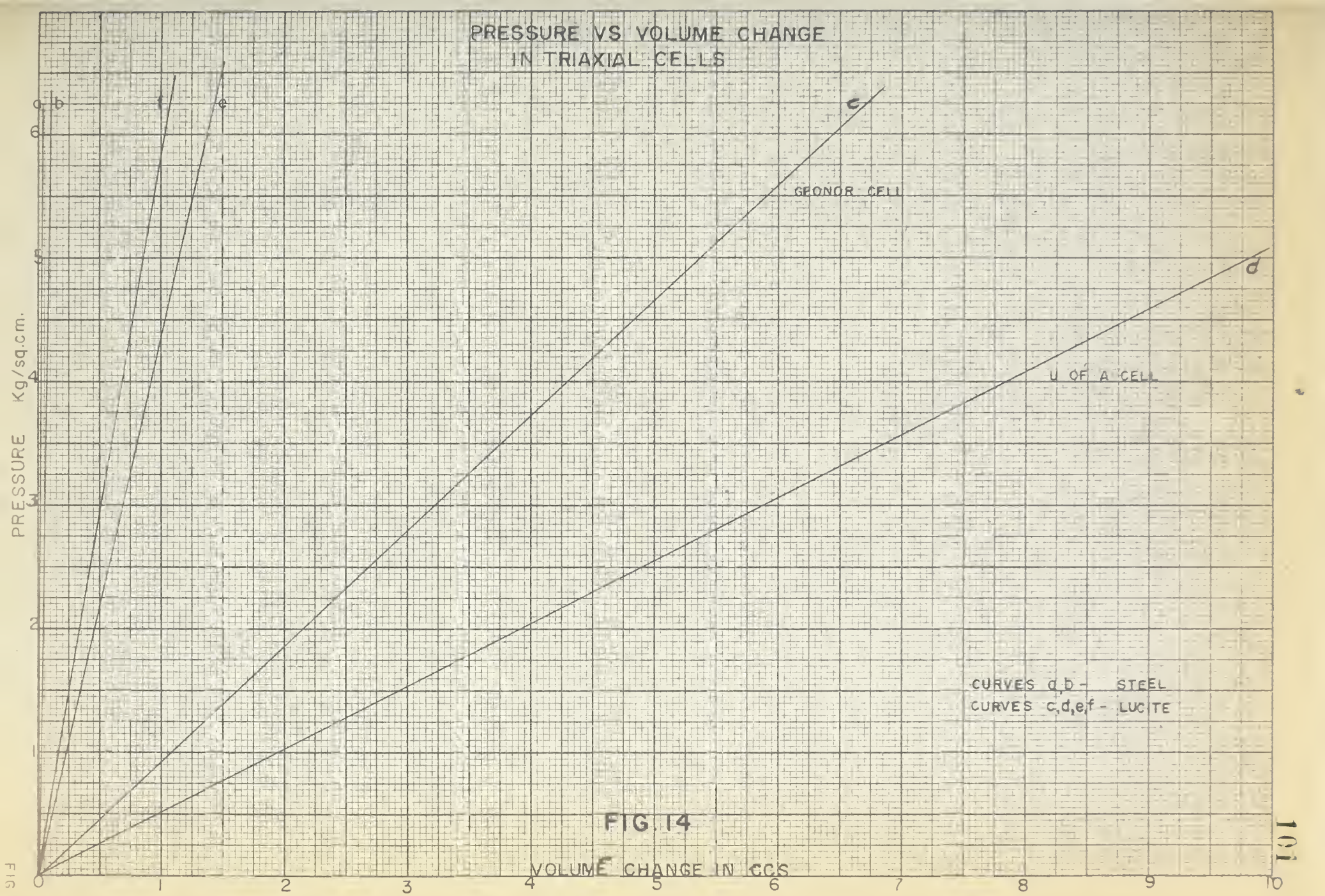

PRESSURE VS VOLUME CHANGE
IN TRIAXIAL CELLS
a, b
c
d
e
f
GEONOR CELL
U OF A CELL
CURVES a, b - STEEL
CURVES c, d, e, f - LUCITE
PRESSURE Kg/sq.cm.
VOLUME CHANGE IN CCS
0
1
2
3
4
5
6
7
8
9
10

FIG 14

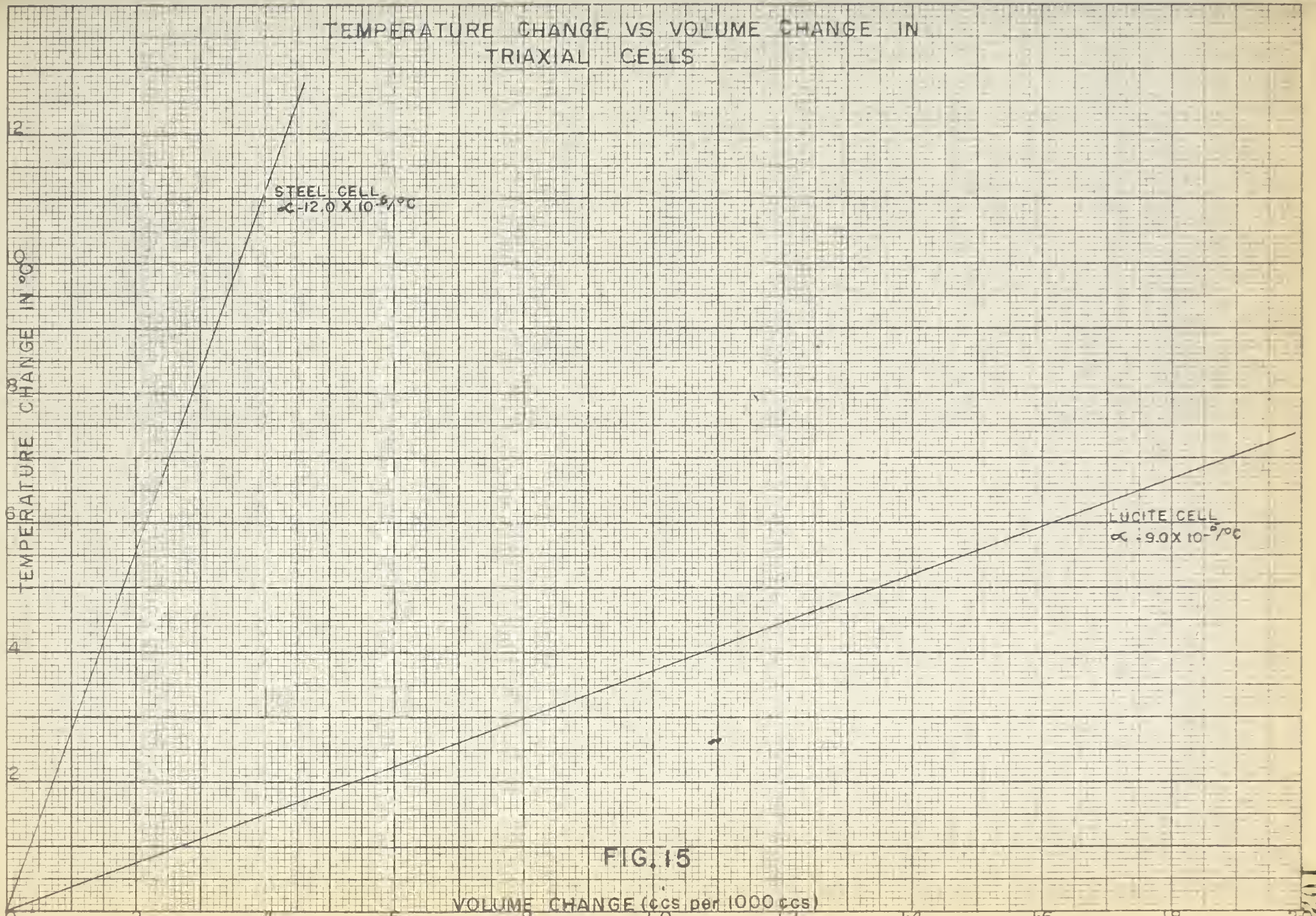
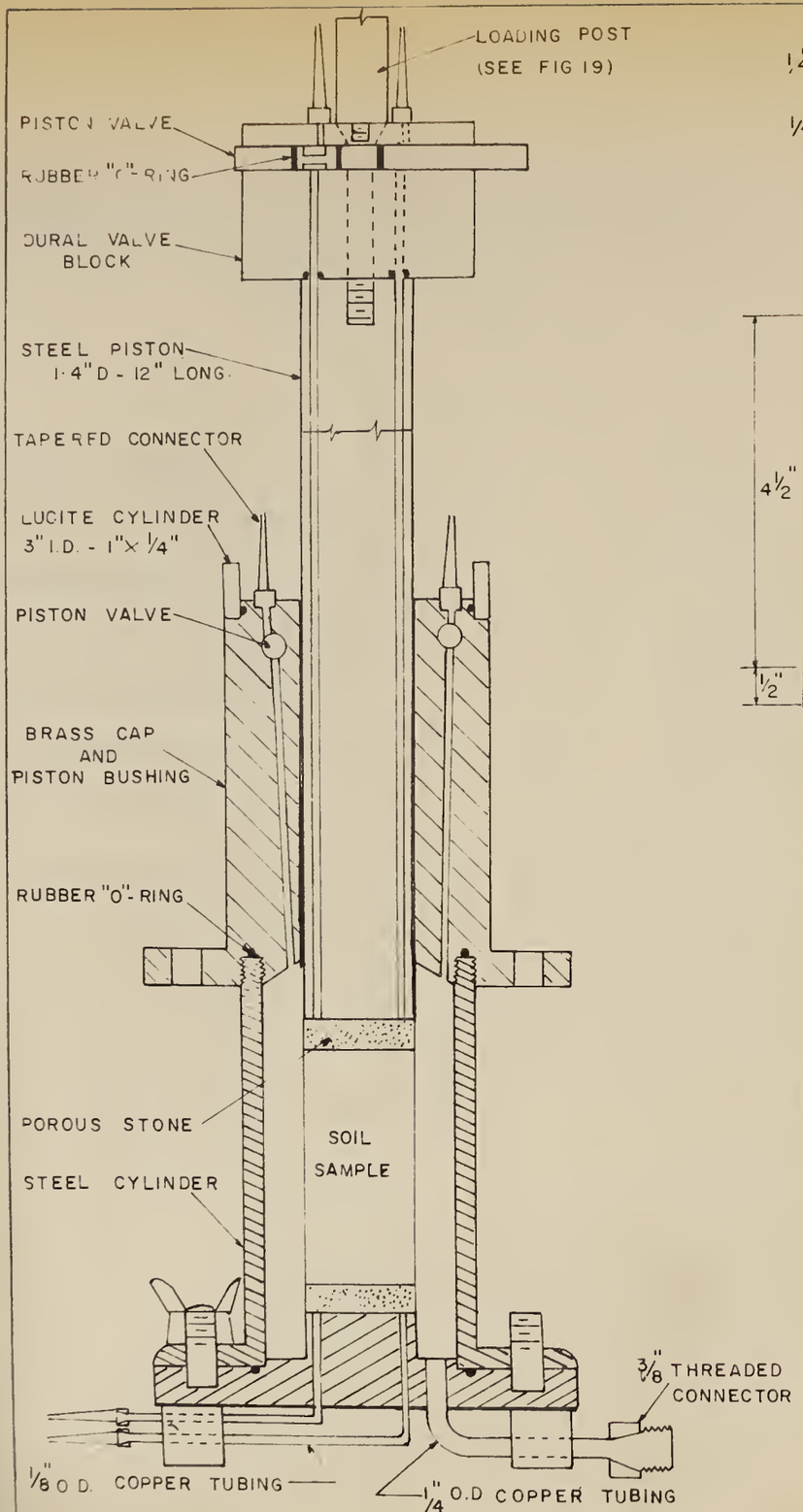
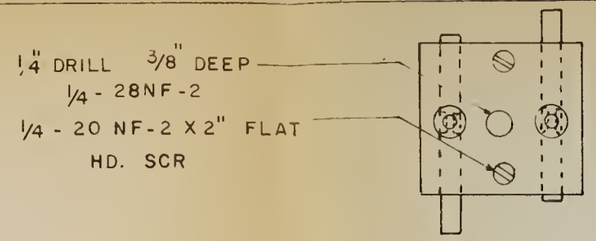

TEMPERATURE CHANGE VS VOLUME CHANGE IN TRIAXIAL CELLS
STEEL CELL
α -12.0 X 10⁻⁶/°C
LUCITE CELL
α -9.0 X 10⁻⁵/°C
TEMPERATURE CHANGE IN °C
VOLUME CHANGE (ccs per 1000 ccs)
0
·2
·4
·6
·8
1·0
1·2
1·4
1·6
1·8
2·0

FIG. 15

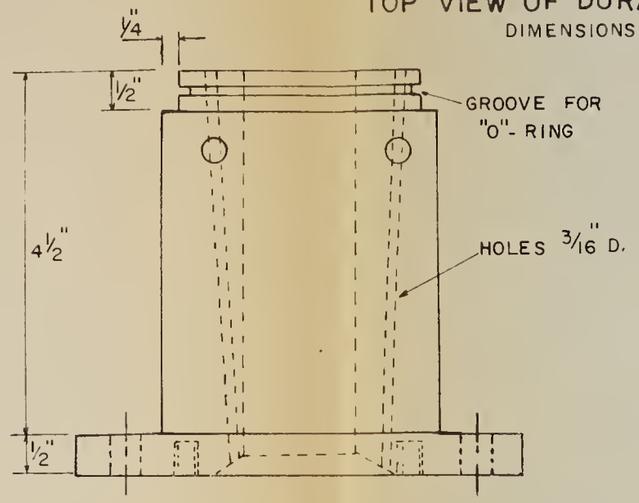
FIG 15



ALL TUBING MUST BE SILVER SOLDERED TO THE BASE INSIDE & OUTSIDE



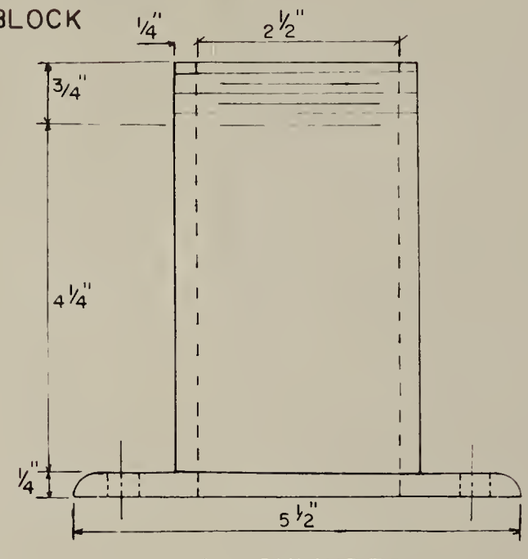
TOP VIEW OF DURAL VALVE BLOCK
DIMENSIONS 2"X 2"X 1½"



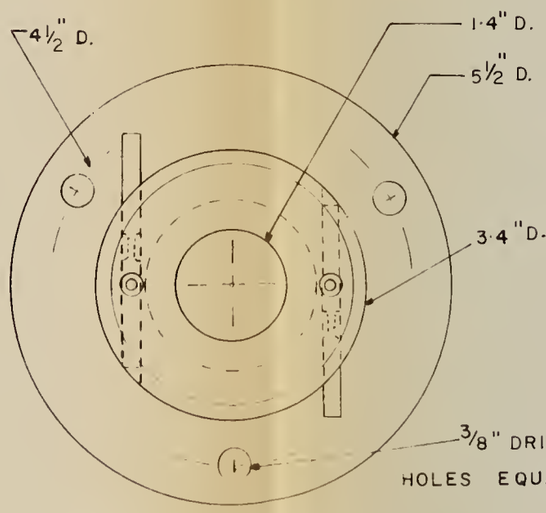
NOTE. PISTON MUST BE LAPPED TO FIT BUSHING ZERO TOLERANCE

MATERIAL FOR BRASS CAP MUST BE FORGED BRASS.

SURFACES OF THE STEEL CYLINDER MUST BE SMOOTH

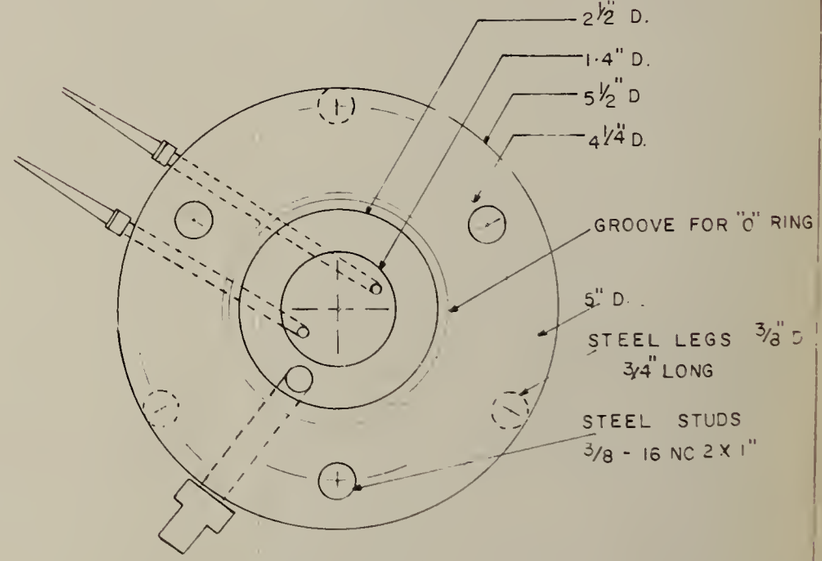


STEEL CYLINDER



FORGED BRASS CAP

FLARE THE ENDS OF THE COPPER TUBING AND SILVER SOLDER TO CONNECTORS.



STEEL BASE

FIG 16
THE STEEL TRIAXIAL CELL

SCALE - HALF SIZE MAY 17, 1961

DESIGNED & DRAWN BY - H O

APPENDIX II

Design of the Apparatus for Measuring Swelling Pressure in a Vertical Direction

Length of Sample	3.0"
Maximum allowable expansion of sample	.001" = .033%
Maximum expected swelling pressure	= 1.5 Kg/sq.cm.
Pressure due to weight of piston	= 0.28 Kg/sq.cm.
Design swelling pressure	= 1.22 Kg/sq.cm.
Total design load (based on a 10 sq.cm. area)	= 12.2 Kg = 26.64 lb.

Design for 30 lb. load

The design load will act on a simply supported aluminum beam having a span (center to center of supports) of 5". E for aluminum = 10×10^6 p.s.i. For a simply supported Aluminum Beam under a load of 30 lbs. and a span of 5", the moment of inertia required to allow a maximum deflection at the center of 0.001" is,

$$I = \frac{Pl^3}{48 \delta E}$$

$$I = \frac{30 \times 125}{48 \times 10^4}$$

$$I = .00783 \text{ in}^4$$

Let cross section of beam be rectangular -- b = 1/2"

$$I = \frac{bt^3}{12}$$

$$t = \sqrt[3]{\frac{12I}{b}}$$

$$= \sqrt[3]{\frac{12 \times 7.83 \times 10^{-3}}{5 \times 10^{-1}}}$$

$$= .575"$$

Make beam 1/2" wide and 0.6" thick.

$$I = 90 \times 10^{-4} \text{ in}^4$$

Check on Aluminum Beam 0.5 x 0.6 x 5" long.

Assume pressure of 0.05 Kg/cm^2

Equivalent load on a 10 sq.cm. area = 0.5 Kg
= 1.1 lb.

$$\text{Strain } \frac{\delta L}{L} = \frac{PL^2}{48EI}$$

$$= \frac{1.1 \times 25}{48 \times 90 \times 10^3}$$

$$= 6.37 \text{ micro inches/inch}$$

The strain gauge indicator is graduated at every ten micro-inches of strain so that readings can be made to the nearest pound load or 0.05 Kg/cm^2

Assume maximum load acting on beam

Load on Beam = 30 lbs. $I = 90 \times 10^{-4}$ in.^{4}

$$\text{Strain } \delta L/L = \frac{30 \times 25}{48 \times 90 \times 10^3}$$

$$= 174 \text{ micro inches/inch.}$$

Maximum allowable strain ------200 micro inches/inch

Maximum Load eventually put on bar ---- 40.2 Kg
---- 88.4 lb.

Length of Beam (c to c) during test ---- 5.5 ins.

(Remoulded samples -- Confining Pressure 4.0 Kg/cm^2)

$$\text{Strain } \frac{\delta L}{L} = \frac{88.4 \times 30.25}{48 \times 90 \times 10^3}$$

$$= 619 \text{ micro inches/inch}$$

Maximum elongation of sample
Maximum Deflection of Bar .0034 inches.

Length of Sample = 3.00 inches

Percentage increase allowed 0.11%

Check on the elongation of the supports under expected maximum load.

Material of supports ---- Steel ---- $E = 30 \times 10^6$ p.s.i.

Assume maximum design load acting on the beam

Load on each support ---- 15 lbs.

Round portion of Bar

Length 7"

Diameter 3/8"

Cross sectional area .11 sq.in.

$$\delta = \frac{PL}{AE} = \frac{15 \times 7}{.11 \times 30 \times 10^6}$$

Elongation = 3.18×10^{-5} inches

Rectangular Portion of Bar

Length = 5"

Dimensions 5/8" x 5/8"

Cross sectional area .39 sq. ins.

$$\frac{PL}{AE} = \frac{15 \times 5}{3.9 \times 3.0 \times 10^6}$$

Elongation = 6.4×10^{-6} ins.

Total elongation of the supports under expected maximum load =

38.2×10^{-6} inches

Maximum load eventually put on supports 44.2 lb.

Elongation of Round portion of supports $= \frac{44.2}{15} \times 3.18 \times 10^{-5}$

$= 9.4 \times 10^{-5}$ inches

Elongation of Rectangular portion of supports $= \frac{44.2}{15} \times 6.4 \times 10^{-6}$

$= 1.89 \times 10^{-7}$ inches

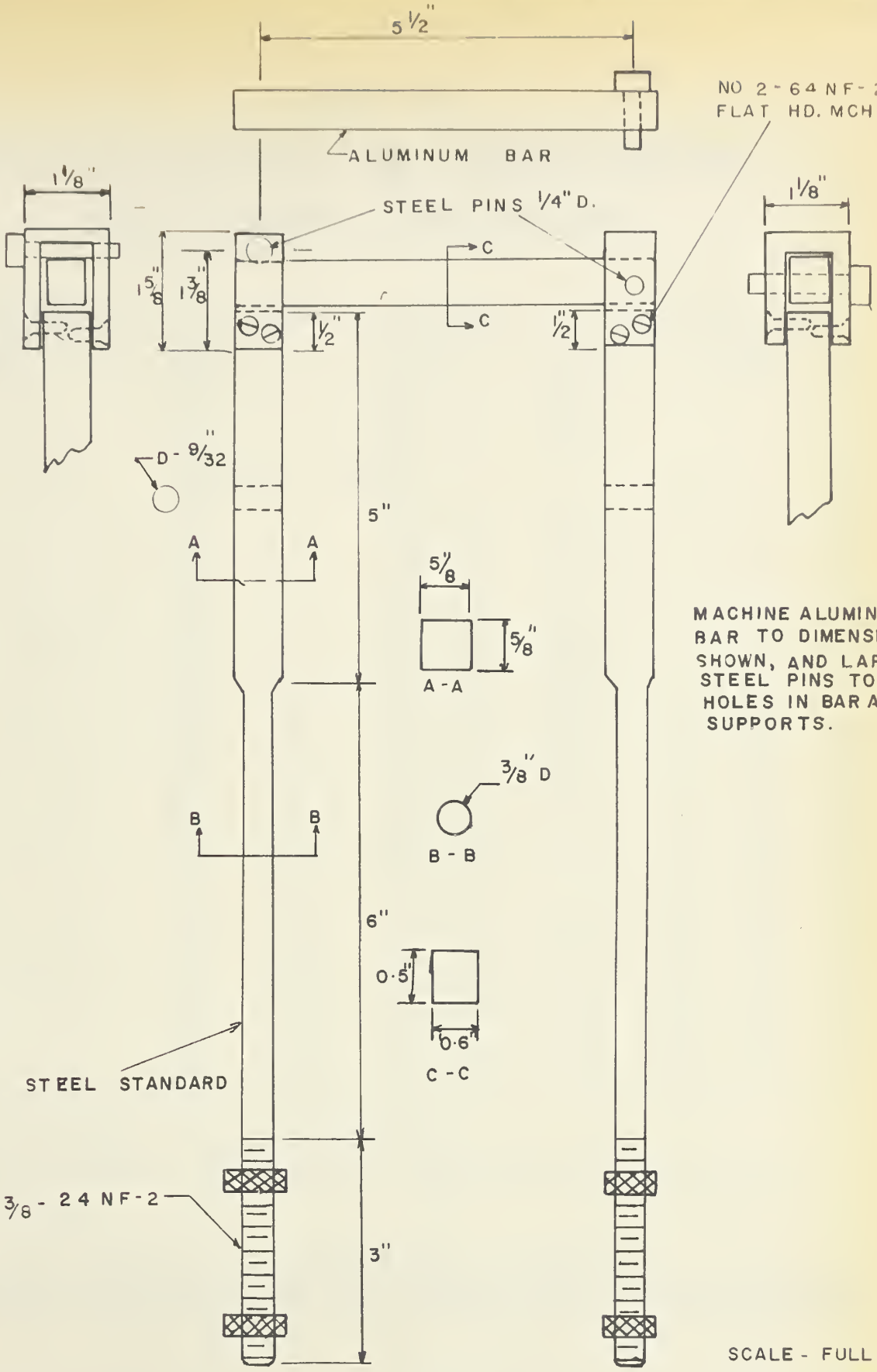
Total elongation of the supports under maximum load encountered $= 112.9 \times 10^{-6}$ inches

This elongation of the supports will not appreciably affect the maximum elongation of the sample computed on the basis of a maximum deflection of the cross bar.

Because the maximum load that eventually acted upon the apparat-

us was about three times as large as the maximum design load, the elongation of the sample was greater than the allowable maximum used for design. This resulted in slightly more than 0.1% change in the length of the sample, and an observed volume increase of 2.1%. Volume increases of 3.4% and 4.2% were observed at lower values of upward force acting on the cross bar. Volume changes of this magnitude may possibly be a source of error. (Finn & Strom. 1958).

The apparatus is shown in Fig. 17, and Tables IX and X give the results of calibrations that were made before and after the tests. These calibration results are plotted in Fig. 18.



MACHINE ALUMINUM BAR TO DIMENSIONS SHOWN, AND LAP STEEL PINS TO FIT HOLES IN BAR AND SUPPORTS.

SCALE - FULL SIZE
MARCH 31, 1962

FIG. 17

PRESSURE MEASURING APPARATUS

TABLE IX

CALIBRATION OF APPARATUS FOR MEASURING SWELLING PRESSURE IN THE VERTICAL DIRECTION

Pressure Kg/sq.cm.	1 st Trial	Diff	2 nd Trial	Diff	3 rd Trial	Diff	Average Diff
0.0	1525	-	1525	-	1525	-	-
.10	1525	-	1525	-	1525	-	-
.20	1525	-	1525	-	1525	-	-
.28	PISTON		RISES				
.30	1528	3	1529	4	1528	3	3
.40	1531	6	1534	9	1534	9	8
.50	1539	14	1539	14	1538	13	14
.80	1560	35	1559	34	1560	35	34
1.00	1570	45	1574	49	1571	46	47
.25	1587	62	1588	63	1588	63	63
.50	1605	80	1603	78	1606	81	80
.75	1620	95	1619	94	1621	96	95
2.00	1638	113	1637	112	1637	112	112
.25	1653	128	1655	130	1651	126	128
.50	1668	143	1669	144	1669	144	144
.75	1684	159	1685	160	1688	163	161
3.00	1703	178	1705	180	1702	177	178
.25	1717	192	1718	193	1718	193	193
.50	1735	210	1735	210	1736	211	210
.75	1752	227	1752	227	1750	225	226
4.00	1767	242	1767	242	1766	241	242
.25	1783	258	1784	259	1785	260	259
.50	1800	275	1798	273	1803	278	275
.75	1815	290	1814	289	1819	294	291
5.00	1831	306	1830	305	1831	306	306

Calibration Made October 31, 1961
Strain Gauge -- Budd C12 - 141 Gauge Factor 2.06
Units of Figures in all columns except the first are micro inches per inch.

N.B. Pressure acts on Piston Area --- 10 sq.cm.
The average values in this chart were used on all remoulded samples.

TABLE X

CALIBRATION OF APPARATUS FOR MEASURING SWELLING PRESSURE IN A VERTICAL DIRECTION

Pressure Kg/sq.cm.	1 st Trial	Diff	2 nd Trial	Diff	Average Diff
0.0	1490	-	1490	-	-
.10	1490	-	1490	-	-
.20	1490	-	1490	-	-
.28	PISTON	RISES			
.30	1494	4	1494	4	4
.40	1500	10	1500	10	10
.50	1508	18	1508	18	18
.60	1512	22	1512	22	22
.70	1520	30	1520	30	30
.80	1525	35	1526	36	35
.90	1532	42	1532	42	42
1.00	1538	48	1540	50	49
.20	1550	60	1556	63	62
.40	1562	72	1564	74	73
.60	1575	85	1578	88	86
.80	1588	98	1590	100	99
2.00	1602	112	1603	113	112
.20	1618	128	1620	130	129
.40	1630	140	1632	142	141
.60	1643	153	1643	153	153
.80	1660	170	1660	170	170
3.00	1673	183	1673	183	183
.20	1688	198	1686	196	197
.40	1702	212	1700	210	211
.60	1715	225	1714	224	224
.80	1730	240	1725	238	239
4.00	1745	255	1743	253	254
.20	1753	263	1752	262	262
.40	1765	275	1765	275	275
.60	1782	292	1781	291	291
.80	1798	308	1797	307	307
5.00	1810	320	1808	318	319

Calibration Made -- March 1, 1962
Strain Gauge Budd C 12-141 Gage Factor 2.06
Units of Figures in All Columns Except the First are Microinches per inch.
N.B. Pressure acts on Piston Area -- 10 sq.cm.

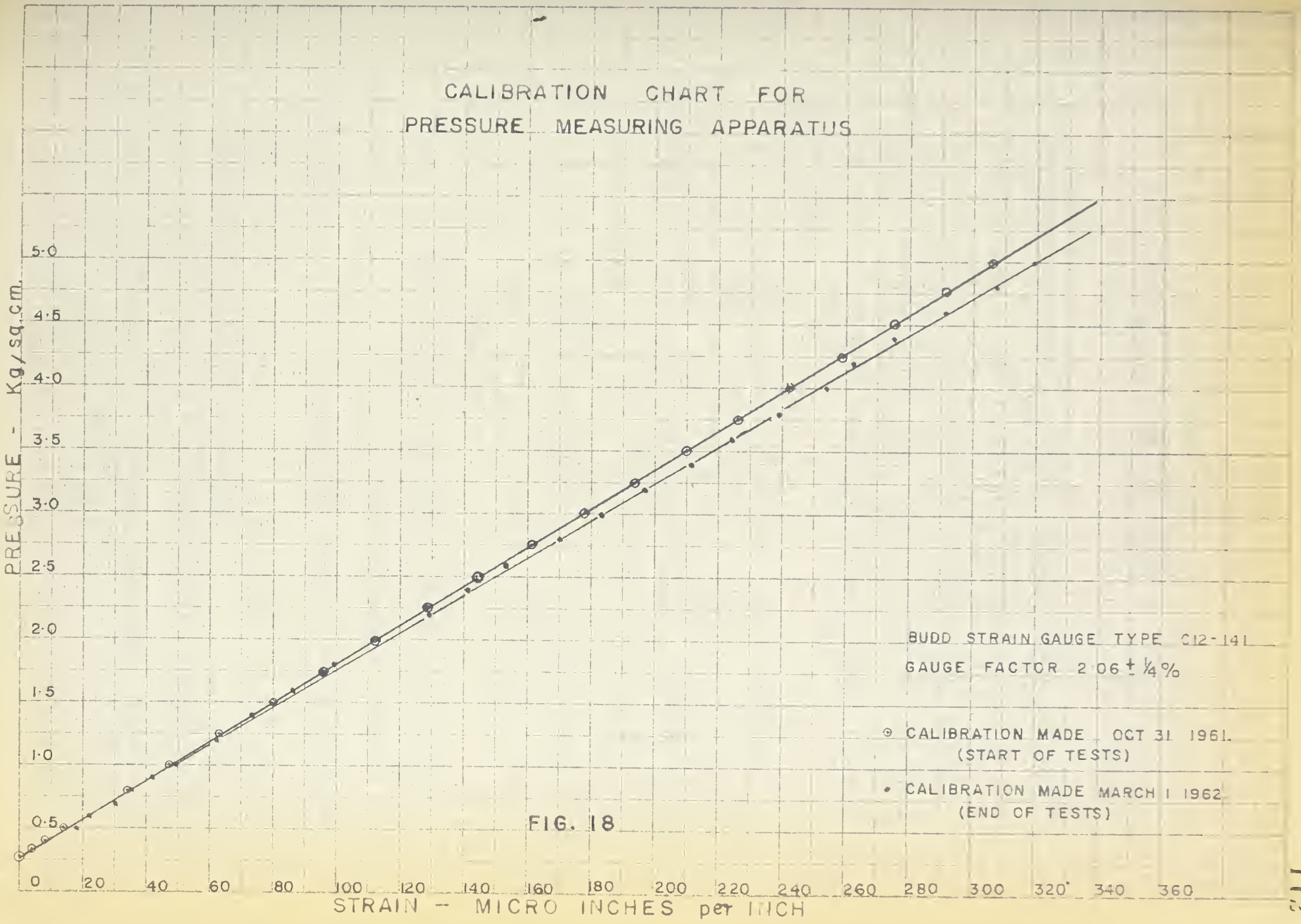

CALIBRATION CHART FOR
PRESSURE MEASURING APPARATUS
PRESSURE - Kg/sq.cm.
5·0
4·5
4·0
3·5
3·0
2·5
2·0
1·5
1·0
0·5
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
STRAIN -- MICRO INCHES per INCH
BUDD STRAIN GAUGE TYPE C12-141
GAUGE FACTOR 2·06 ± ¼%
⊙ CALIBRATION MADE OCT 31 1961.
(START OF TESTS)
• CALIBRATION MADE MARCH 1 1962
(END OF TESTS)

FIG. 18

FIG 18

APPENDIX III

DETAILED TEST PROCEDURE

a. Preparation of Remoulded Samples. About 4.0 Kilograms of the soil recovered from a depth of seven feet, and an equal quantity recovered from a depth of nine feet, were air dried and then ground fine enough to pass the No. 60 sieve. Both quantities were thoroughly mixed, and the total batch was divided into samples each weighing about 175 grams.

For remoulding, each sample was mixed with 100% distilled water (the liquid limit) and allowed to soak overnight. The soil was then placed in a lucite mould 1.75 inches in diameter and 5.62 inches long, and allowed to consolidate under a pressure of 0.4 Kg/sq.cm., and then under a pressure of 1.6 Kg/sq.cm. Filter paper strips were placed along the length of the mould to help in speeding up the consolidation process. (See Plate IIIa).

After consolidation under the higher pressure was complete, the samples were removed from the mould, wrapped in aluminum foil and waxed. The water content of the samples at this stage varied between 60 and 65 per cent.

b. Procedure for Constant Volume Tests.

1. The wax and aluminum foil was removed from the sample, and it was trimmed to a diameter of 1.4 inches, and a length of three inches.

2. The dimensions of the sample were taken. The length was taken as the average of two measurements, and the diameters at the top, the middle and the bottom were also taken as the average of two measurements. The average cross-sectional area was computed as

$$A_v = \frac{A_t + 2A_m + A_b}{4}$$

where A_v - The average area of the sample
A_t - The area at the top of the sample
A_m - The area at the middle of the sample
A_b - The area at the bottom of the sample

The volume of the sample was taken as

Average length x Average cross sectional area

3. Saturated wool wicks were placed in the sample. These wicks extended from the bottom to within one centimeter of the top. One wick was placed at the center, while the other four were evenly spaced about the centrally placed wick, at a distance half way between the center and the circumference.

4. The sample was weighed to the nearest milligram.

5. A burette filled with distilled water was attached to the base of the steel cell, and air was flushed out of the pore pressure lines. A saturated porous ceramic stone was placed on the pedestal of the cell base, and held in place by a piece of 1 1/4 inch diameter rubber tubing. Flushing of the lines was continued until all air had been removed, then the water outlet was closed.

6. The piston while in place in the brass bushing, had its lower end

thoroughly cleaned. A saturated porous ceramic stone was attached to this end, by means of a piece of rubber tubing, and water was circulated through the pore pressure lines until all air had been removed. The water outlet was then closed.

7. A saturated filter paper disc (Watman No. 54) was placed on the lower porous stone, and two rolled membranes were placed over the pedestal.

8. Rubber "O" rings 1 1/8 inches in diameter were stretched over the brass split ring (Fig. 19), two above the handle and two below. The brass ring was placed over the pedestal and allowed to remain sitting on the base of the cell.

9. The cell support (Fig. 20 and Plate III-b) was placed around the base, and the upper part of the cell was placed on it.

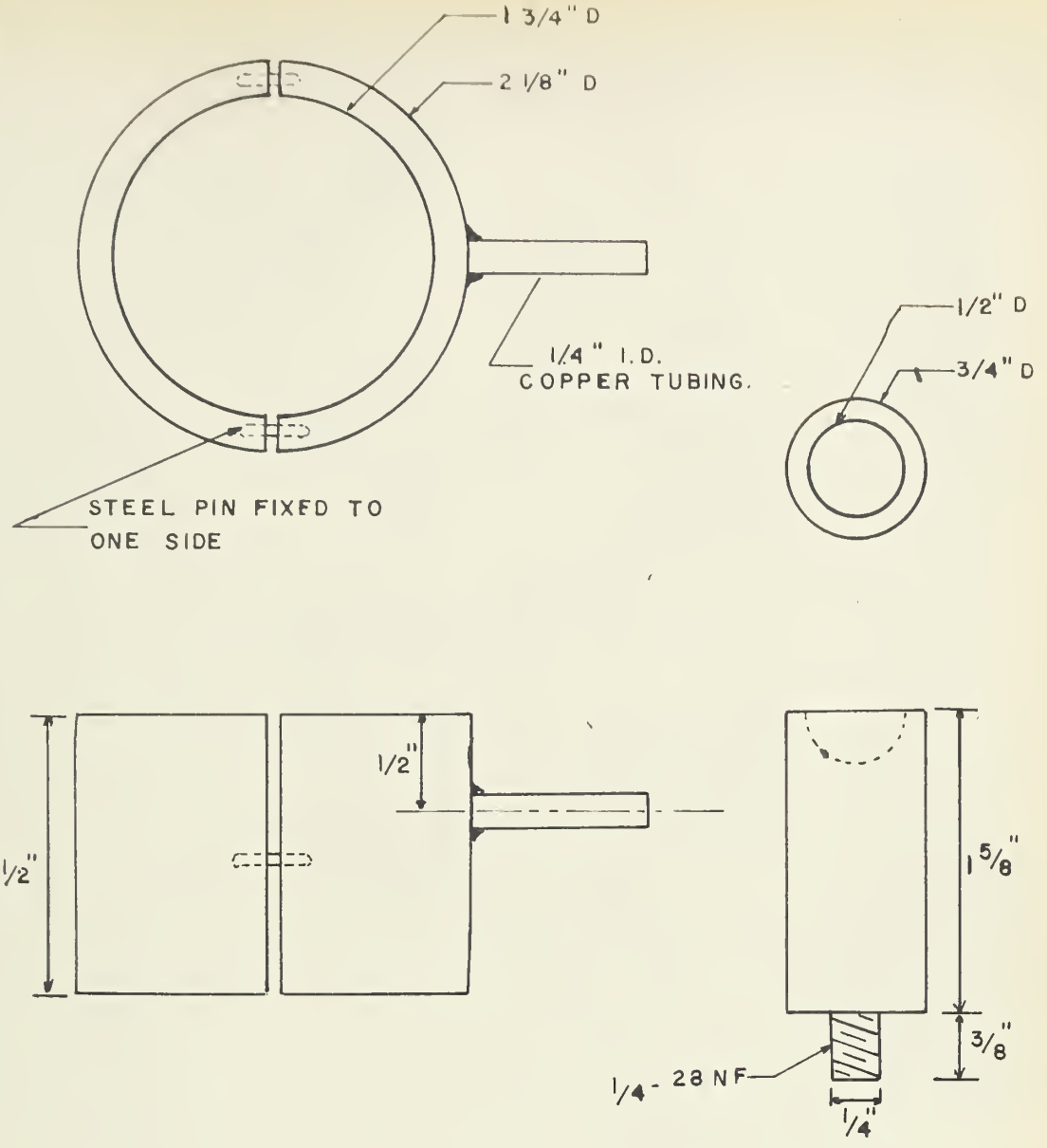
10. The holes in the flange of the steel cylinder were aligned with the studs in the base, and the piston was turned to permit easy mounting of the cross beam for measuring vertical swelling pressure.

11. The water inlets to both the top and bottom porous stones were closed, and excess water on their surfaces was wiped away. The piston was raised an inch or two, and the trimmed sample was placed on the lower porous stone. A saturated filter paper disc was placed on the top of the sample, and the piston was gradually lowered until the upper porous stone rested on the sample.

(a)— APPARATUS FOR REMOULDING SAMPLES

(b) CELL SUPPORT AND BRASS SPLIT RING

PLATE III



BRASS SPLIT RING

DURAL LOADING POST

SCALE - FULL SIZE
MARCH 31, 1962

FIG. 19

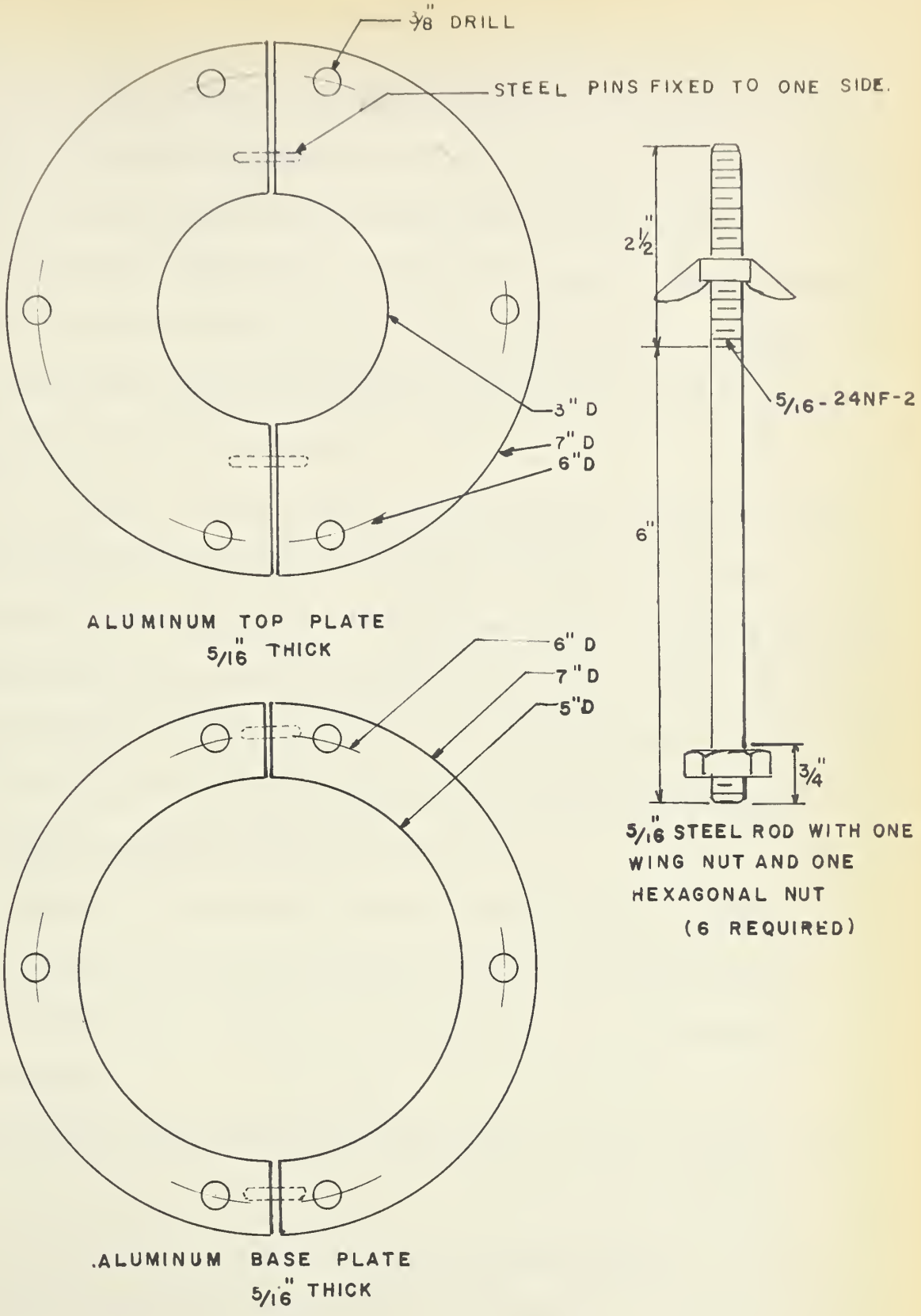


FIG. 20

CELL SUPPORT

SCALE - HALF SIZE
MARCH 31, 1962

12. One rubber membrane was rolled upwards until it reached the piston, and a film of silicone grease* was spread over this membrane. The second membrane was then rolled upward over the first.

13. The two "O" rings below the handle of the brass ring were snapped on to the pedestal to hold the membranes firmly in place. Similarly the remaining two rings were snapped onto the piston. The brass ring was then separated and removed.

14.** The weight of the upper part of the cell was supported by one man, a second man held the piston stationary and erect, while a third man removed the supporting stand. With the piston held stationary and erect, the upper part of the cell was gradually moved downwards into position on the cell base. The wing nuts were put on and tightened.

15. The cell was filled with de-aired water from the bottom, while air escaped through the holes in the brass cap.

16. When the cell was filled, a line was attached to the lower hole in the brass cap and oil was pumped into the cell, until it began flowing from the other hole. The oil line was then removed, and the holes were capped.

17. The water level in the burette was brought as low as possible, and

* The grease used was Dow Corning High Vacuum grease.

** Extreme care must be exercised during this step.

the initial volume recorded. A measurement was taken of the height of the piston above the top of the cell.

18. The cell was placed on the platform machine, and after balancing, the loading yoke was brought in contact with the loading post on top of the piston. (Plate IV-a)

19. A calculation was made of the load required on the piston to produce a vertical stress equal to the confining pressure of 7.00 Kg/ sq. cm.

20. The drainage valve to the burette was opened, the timer was started and confining and vertical pressures were gradually increased to 7.00 Kg/sq. cm.

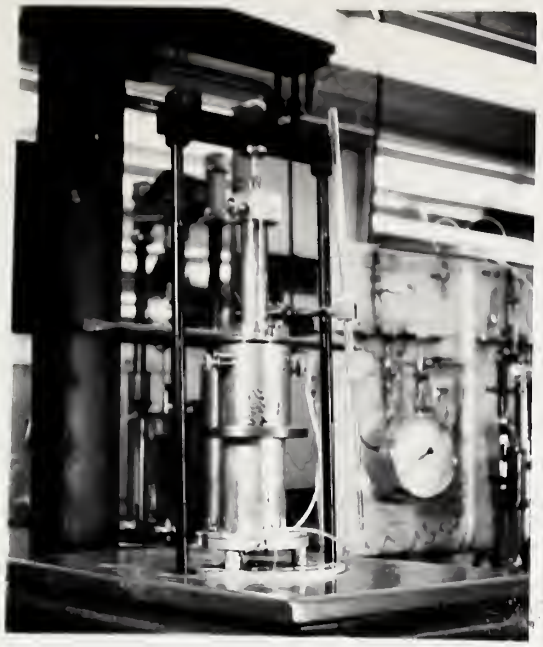
21. Readings were taken and plots were made of volume change versus the logarithm of time.

22. Upon completion of consolidation, the drainage valve to the burette was closed, and the confining and vertical pressures were reduced to zero. The cell was removed from the platform machine.

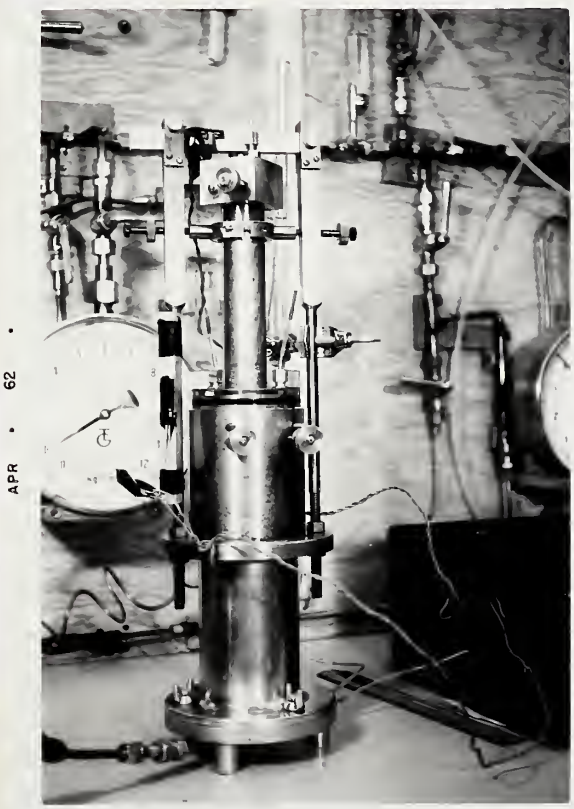
23. The device for measuring swelling pressure in the vertical direction was mounted on top of the cell, and a zero reading was taken on the strain gauge indicator. (Plate IV-b)

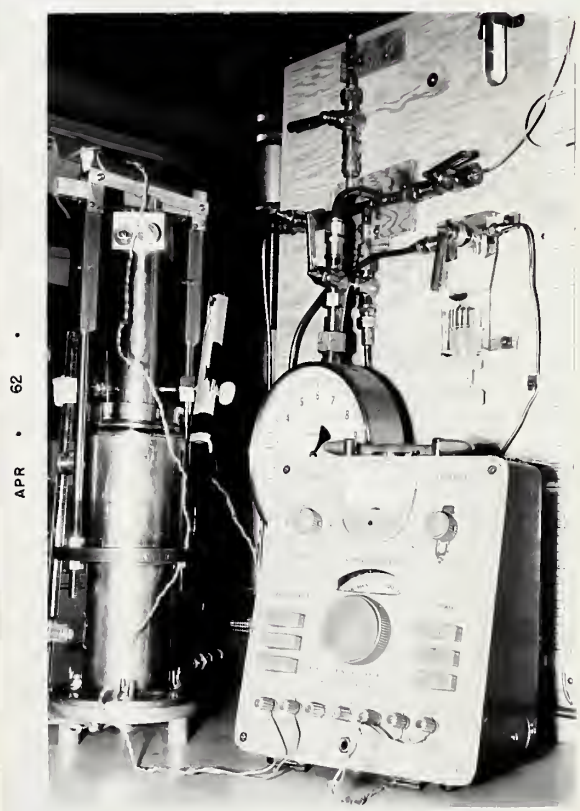
24. The pore pressure line was connected to the outlet on the top of the piston.

25. The confining pressure was increased to the desired initial value,



(a) SAMPLE CONSOLIDATING IN THE PLATFORM MACHINE





(b) APPARATUS FOR MEASURING VERTICAL SWELLING PRESSURE

PLATE IV

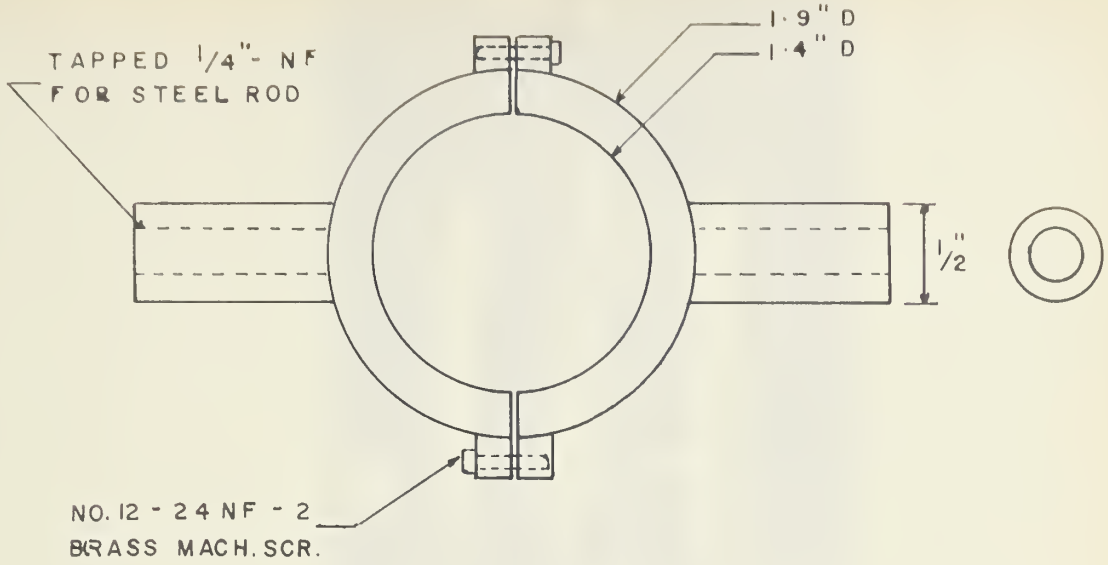
the bypass valve of the null indicator in the line to the constant pressure device was closed, and the same valve of the null indicator in the pore pressure line was also closed.

26. The drainage valve to the burette was opened, the timer was started, and readings were made at frequent intervals of values of horizontal and vertical swelling pressure, pore pressure and water volume in the burette. Values of vertical swelling pressure were read from the calibration chart for the crossbeam (Fig. 18), by using the difference between the zero reading and any subsequent reading taken on the indicator. Values of horizontal swelling pressure were read on a Bourdon gauge or mercury manometer as the pressure required to keep the null indicator balanced.

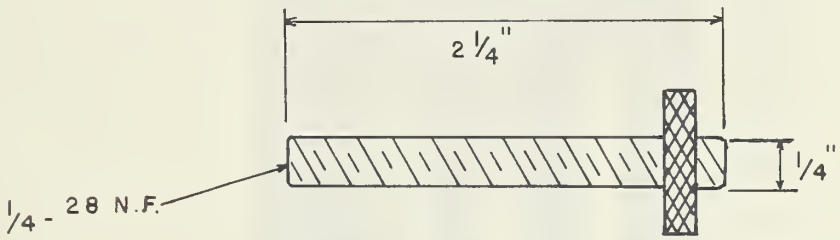
27. When the sample showed no further tendency to increase either the horizontal or vertical swelling pressure, the drainage and pore pressure valves were closed, and a measurement was taken of the height of the piston above the top of the cell. The clamp (Fig. 21) was then mounted to keep the piston stationary.

28. The cross beam was removed, the cell was placed in the compression machine, and a load (measured by a proving ring) was applied to the piston. This load was made equivalent to the upward force which was measured on the cross beam. (Plate V-a)

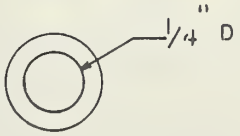
29. Water was flushed through both the top and bottom pore pressure

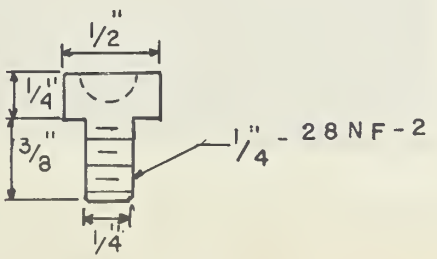


BRASS PISTON CLAMP



STEEL ROD AND KNURLED NUT

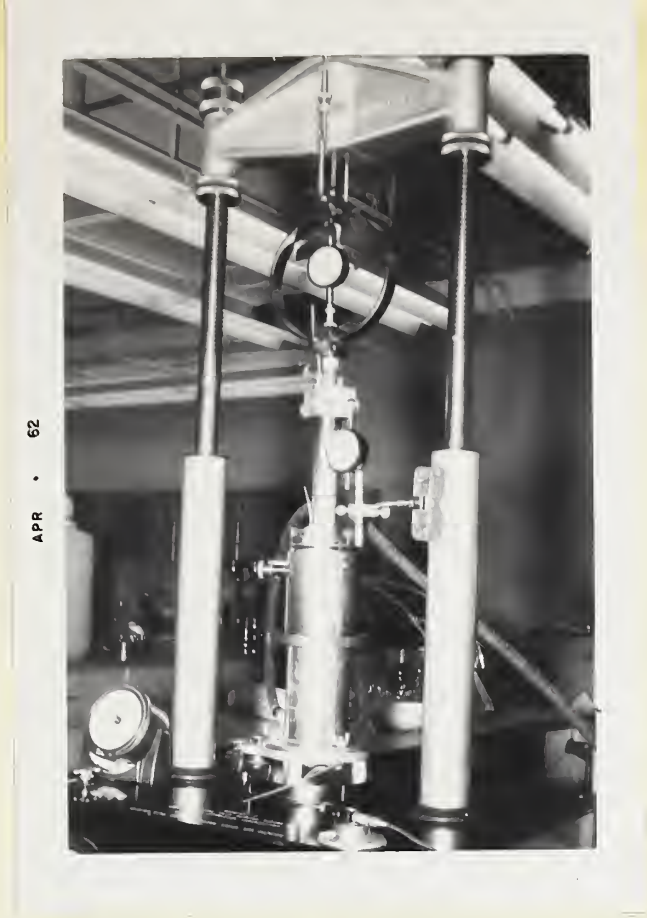




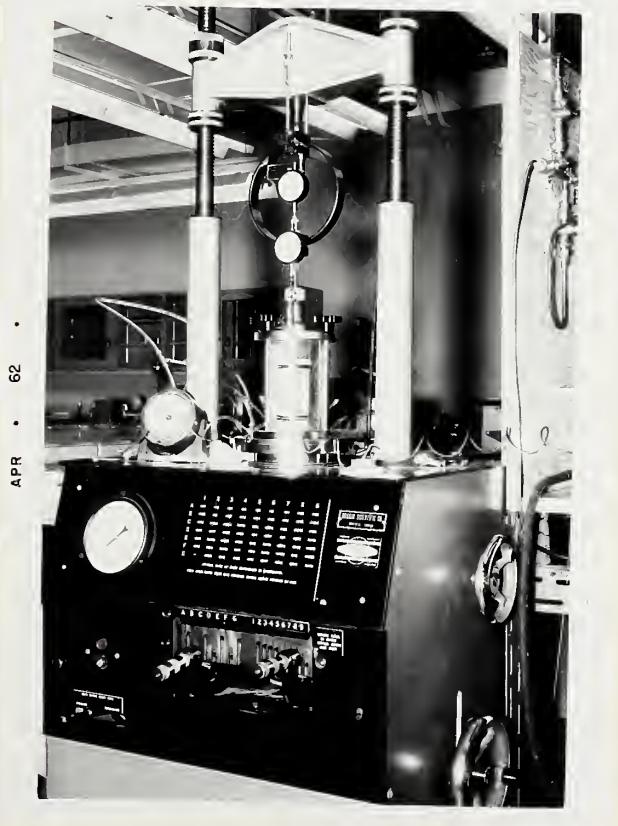
LOADING POST

SCALE - FULL SIZE
MARCH 31, 1962

FIG. 21



(a) THE STEEL CELL IN THE COMPRESSION MACHINE



(b) THE STANDARD CELL IN THE COMPRESSION MACHINE

PLATE V

lines until all air was removed. The valves in the top pore pressure lines were closed, and connections were made to the bottom pore pressure lines for measurement of pore pressure during the compression part of the test.

30. Compression of the sample was carried out as for a consolidated undrained triaxial test. Average area was computed as given in Lambe (1951).

31. After the sample failed, it was removed from the cell and the weight obtained to the nearest milligram. Its volume was found by immersion in mercury, and then it was dried to constant weight at 110° C. From the data obtained determinations were made of water content, void ratio and degree of saturation, both before and after consolidation.

c. Procedure for Standard Test -- Steel Cell.

1. Steps 1 to 21 above were carried out, except that under step 6, a solid disc was attached to the piston instead of a porous ceramic stone, and no circulation of water was required in the piston.

2. At the end of consolidation, the drainage valve was closed and a computation was made to find the load required to give a vertical pressure on the sample, which would be equal to the confining pressure required for the test.

3. Both horizontal and vertical pressures were simultaneously re-

duced to the value required for the test. The drainage valve was opened, and the timer started. A plot was made of volume change versus the logarithm of time.

4. When swelling was complete, measurements of the height of piston were taken, and the net volume change in the sample was found.

5. The clamp (Fig. 22) was attached to the piston to keep it stationary, and the cell was removed from the platform machine to the compression machine. Here a load was applied to the piston, which was equal in magnitude to the load that acted on the piston while it was in the platform machine.

6. The clamp was removed.

7. Water was flushed through the bottom pore pressure lines, and connections were made for pore pressure measurements during the compression part of the test.

8. Steps 30 and 31 above were carried out.

d. Sample Data Sheets and Calculations.

The calculations for all tests were made following the method outlined in Lambe (1951). The sample data sheets are for the Constant Volume test which had an initial confining pressure of 1.50 Kg/sq.cm., and a final confining pressure of 2.50 Kg/sq.cm. With the exception of the Strain Gauge Data, the data sheets are typical for all tests carried out. Complete laboratory data sheets are given under separate binding.

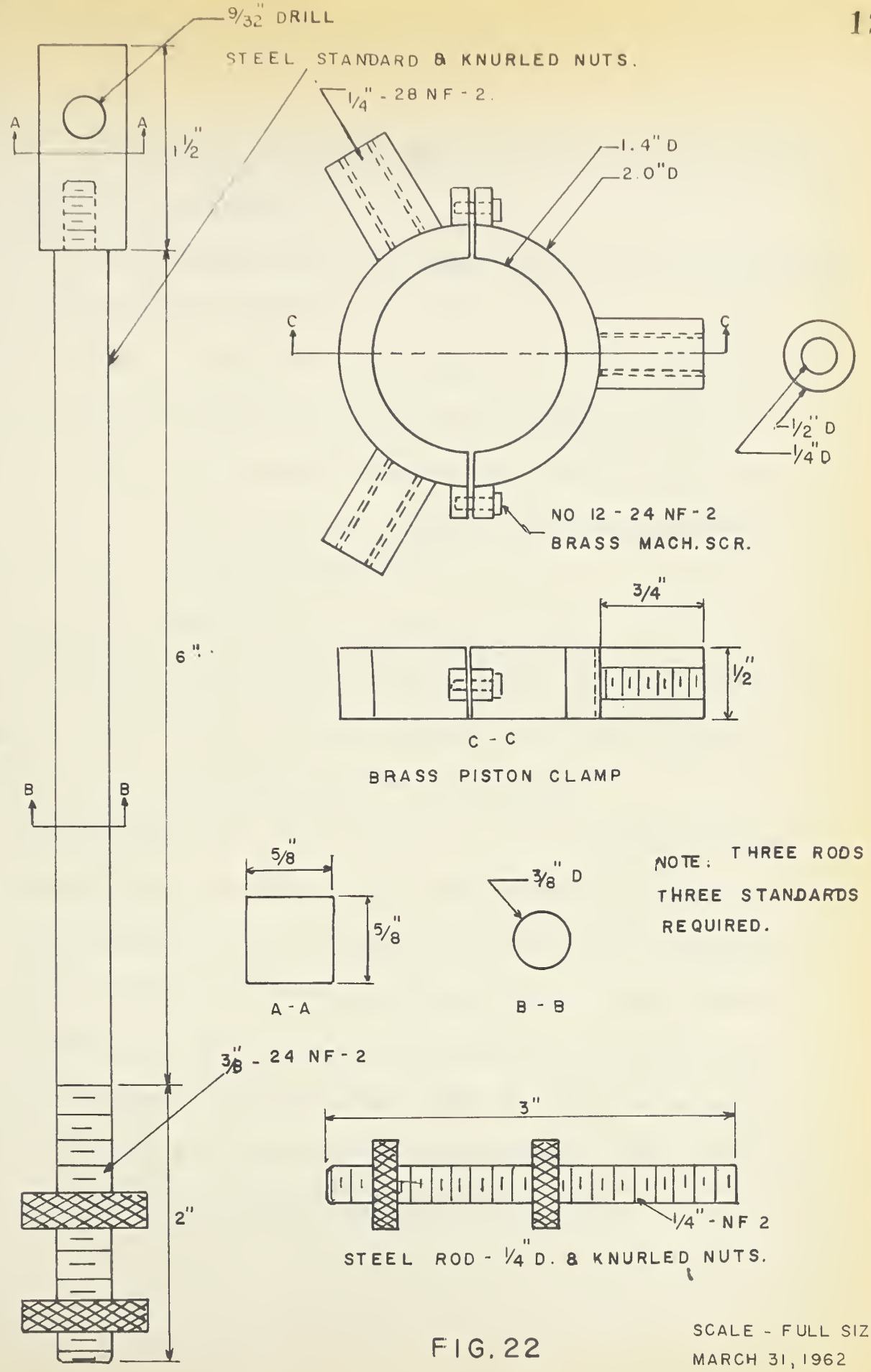


FIG. 22

SCALE - FULL SIZE
MARCH 31, 1962

PISTON CLAMP AND STANDARD

Most of the calculations on Sheet 1 and 4 are self explanatory and need no elaboration.

The symbols used in computing the average area of the sample at the end of the consolidation period are as follows:

Ac. Average area of the sample at the end of consolidation

Vo. Initial volume of the sample found from dimensions.

Vc. Total change in volume of the sample (from initial to final conditions), measured by the change in water volume in the burette.

Lo. Initial average length of the sample.

Lc. Total change in length of the sample (from initial to final conditions), found by observing the Height of the Piston above the top of the cell.

Consolidation Data is shown on Sheet 2. In the data under "Consolidation at Constant Volume" the horizontal swelling pressure is the recorded value of σ_3. The values recorded in the last column are found by subtracting the zero reading on the Strain Gauge Indicator from the reading listed under "Strain Gauge Reading", and using this value in the calibration chart, Fig. 18. The total upward force on the piston due to swelling was calculated as the area of the piston multiplied by the values recorded in the last column.

Calculations for the Triaxial Compression Test were as follows: (Using conditions at failure).

$$\% \text{ Strain} \quad \frac{\text{Change in length due to compression } (\Delta L)}{Lo - Lc} = \frac{(135-100)}{2.82} \times 100$$

$$= 1.25$$

$$\text{Corrected Area } (A_{co}) = \frac{Ac}{1 - \frac{\Delta L}{(Lo - Lc)}} = \frac{8.04}{(1 - .0125)} = 8.14 \text{ sq.cm.}$$

Total Load — Stress dial reading multiplied by Proving ring factor, plus weight of piston

$$= (445 \times .101) + 2.8$$

$$= 47.75 \text{ Kg.}$$

σ_1 Total — Total load divided by corrected area

$$= \frac{47.75}{8.14}$$

$$= 5.86 \text{ Kg/sq.cm.}$$

$\bar{A}$ — Change in Pore Pressure divided by change in deviator stress

$$= \frac{(0.30 - 0.02)}{(5.56 - 2.20)}$$

$$= 0.083$$

It would be of interest to know the percentage error present in the values of c and ϕ, but because of the presence of errors which it is not possible to estimate, and because of the graphical procedure used to obtain c and ϕ, a mathematical approach to the assessment of a percentage error is quite difficult. However, it is possible to compute values of c and ϕ for each test series, and to note the closeness of agreement between the computed values and the values

found from Mohr circles.

The method used is outlined by Balmer (1952) and employs the theory of least squares. A comparison between the results obtained in each case is given below.

Method used	Std. Tests		Constant Volume Tests			
			P_s included		P_s excluded	
	c	∅	c	∅	c	∅
Mohr Circles	0.60	11.5	0.25	19.0	0.70	20.0
Least Squares Theory	0.51	12.4	0.15	20.6	0.73	18.2

Effective values of c and ∅
c in Kg/sq.cm.
∅ in Degrees

The very close agreement indicates that the values of c and ∅ obtained from the Mohr plot are reliable.

SAMPLE DATA SHEET - 1

TEST Constant Volume Test
SAMPLE Remoulded Morrin Clay
DATE January 20, 1962

Length of Sample

(1) 2.930 ins. (2) 2.925 ins. Av. 2.927 ins. = 7.43 cms.

Diameter of Sample (inches)

Top	(1)	1.390	(2)	1.400	Av. 1.395	Av.cms.	3.54
Middle	(1)	1.400	(2)	1.410	1.405		3.57
Bottom	(1)	1.425	(2)	1.430	1.427		3.62

Area (sq. cm.) Top(A_t) 9.84 Middle (A_m) 10.00 Bottom (A_b) 10.28

Average Area $\frac{A_t - 2A_m - A_b}{4}$ = 10.03 sq. cm.

Volume of Sample (10.03 x 7.43) = 74.52 sq. cm.

Strain Gauge Data

Zero Reading	445
Reading Before Filling Cell	445
Reading After Filling Cell	445
Reading after applying σ_3	490
Reading at the end of Consolidation	640

Before Compression

Strain Gauge Reading	640
Calculated upward Force(Kg)	32.8
Weight of Piston (Kg)	2.8
Upward force on proving ring	30.0
Required Proving Ring Reading	297

Initial Confining Pressure 1.5 Kg/sq. cm.

Final Confining Pressure 2.50 Kg/sq. cm.

Height of Piston

initial		7.55"
final		7.44"
Change	-	0.11"
	-	0.28cms

Average area of the sample at the start of compression.

$$A_c = \frac{Vo - Vc}{Lo - Lc}$$

$$= \frac{74.52 - 17.0}{7.43 - 0.28}$$

A_c = 8.04 sq. cm.

SAMPLE DATA SHEET - 2

CONSOLIDATION DATA

TEST Constant Volume Test
SAMPLE Remoulded Morrin Clay
DATE January 20, 1962

Consolidation at 7.00 Kg/sq.cm.

Date	Elapsed Time	Burette Reading ccs.	Date	Elapsed Time	Burette Reading ccs.
Jan. 20/62	0 Min	24.5	Jan. 20/62	1 Hr.	14.7
"	1 "	22.6	"	2 Hr.	10.8
"	2 "	22.3	"	4 Hr.	8.0
"	4 "	22.0	"	8 Hr.	7.2
"	8 "	21.0	Jan. 21/62	24 Hr.	5.6
"	15 "	19.5	Jan. 22/62	48 Hr.	5.6
"	30 "	17.0			

Consolidation at Constant Volume Initial σ_3 = 1.5 Kg/sq.cm.

Date	Elapsed Time (Min.)	Burette Reading ccs.	σ_3 Kg/sq.cm	P_p Kg/sq.cm	Strain Gauge Reading	Vertical Pressure Kg/sq.cm
Jan. 22/62	0	5.6	1.50	0.03	490	0.98
"	1	5.6	1.50	0.03	490	0.98
"	2	5.65	1.50	0.03	490	0.98
"	4	5.70	1.50	0.03	495	1.00
"	8	5.8	1.55	0.03	495	1.00
"	15	5.9	1.60	0.03	500	1.20
"	30	6.0	1.65	0.03	510	1.35
"	60	6.2	1.83	0.02	558	2.00
"	90	6.3	2.14	0.02	572	2.25
"	120	6.4	2.48	0.02	585	2.45
"	150	6.45	2.50	0.02	592	2.55
"	180	6.50	2.50	0.02	598	2.60
"	210	6.60	2.50	0.02	600	2.70
"	240	6.60	2.50	0.02	600	2.70
"	270	6.70	2.50	0.02	604	2.73
"	300	6.70	2.50	0.02	604	2.73
"	330	6.75	2.50	0.02	604	2.73
"	360	6.80	2.50	0.02	600	2.68
"	390	6.80	2.50	0.02	600	2.68

SAMPLE DATA SHEET - 2

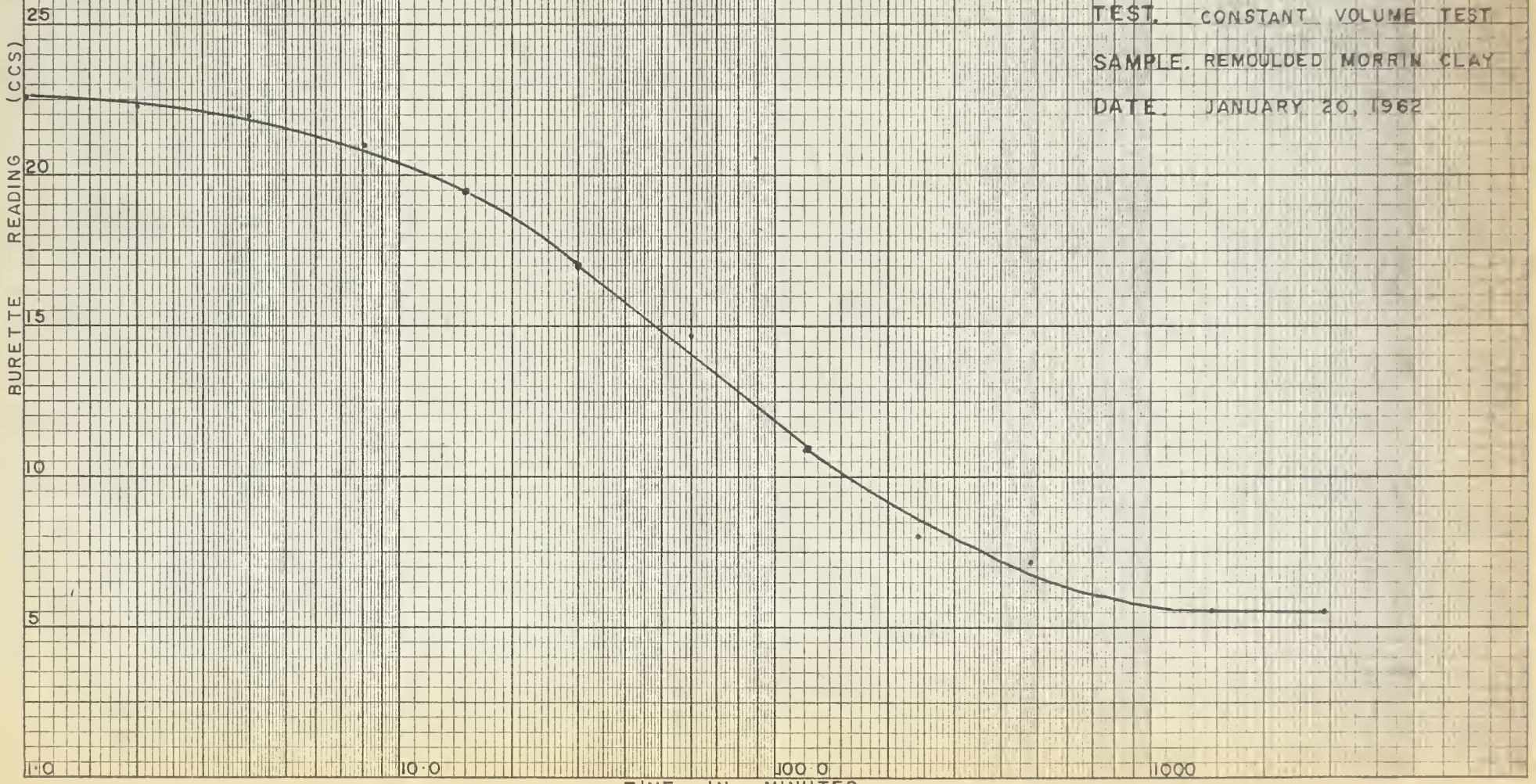
Consolidation at Constant Volume (concluded)

Date	Elapsed Time (Min)	Burette Reading ccs.	σ_3 Kg/sq.cm	P_p Kg/sq.cm	Strain Gauge Reading	Vertical Pressure Kg/sq.cm.
Jan. 22/62	420	6.85	2.50	0.02	600	2.68
"	450	6.90	2.50	0.02	604	2.73
"	480	6.90	2.50	0.02	604	2.73
"	660	7.00	2.50	0.02	604	2.73
"	780	7.20	2.50	0	604	2.73
Jan.23/62	1440	7.35	2.50	0.01	622	3.00
"	1800	7.40	2.50	0.01	640	3.28
Jan. 24/62	2880	7.50	2.50	0.03	640	3.28
Jan. 25/62	4320	7.50	2.50	0.05	640	3.28

SAMPLE DATA SHEET — 2a

CONSOLIDATION AT 7·0 Kg/sq cm

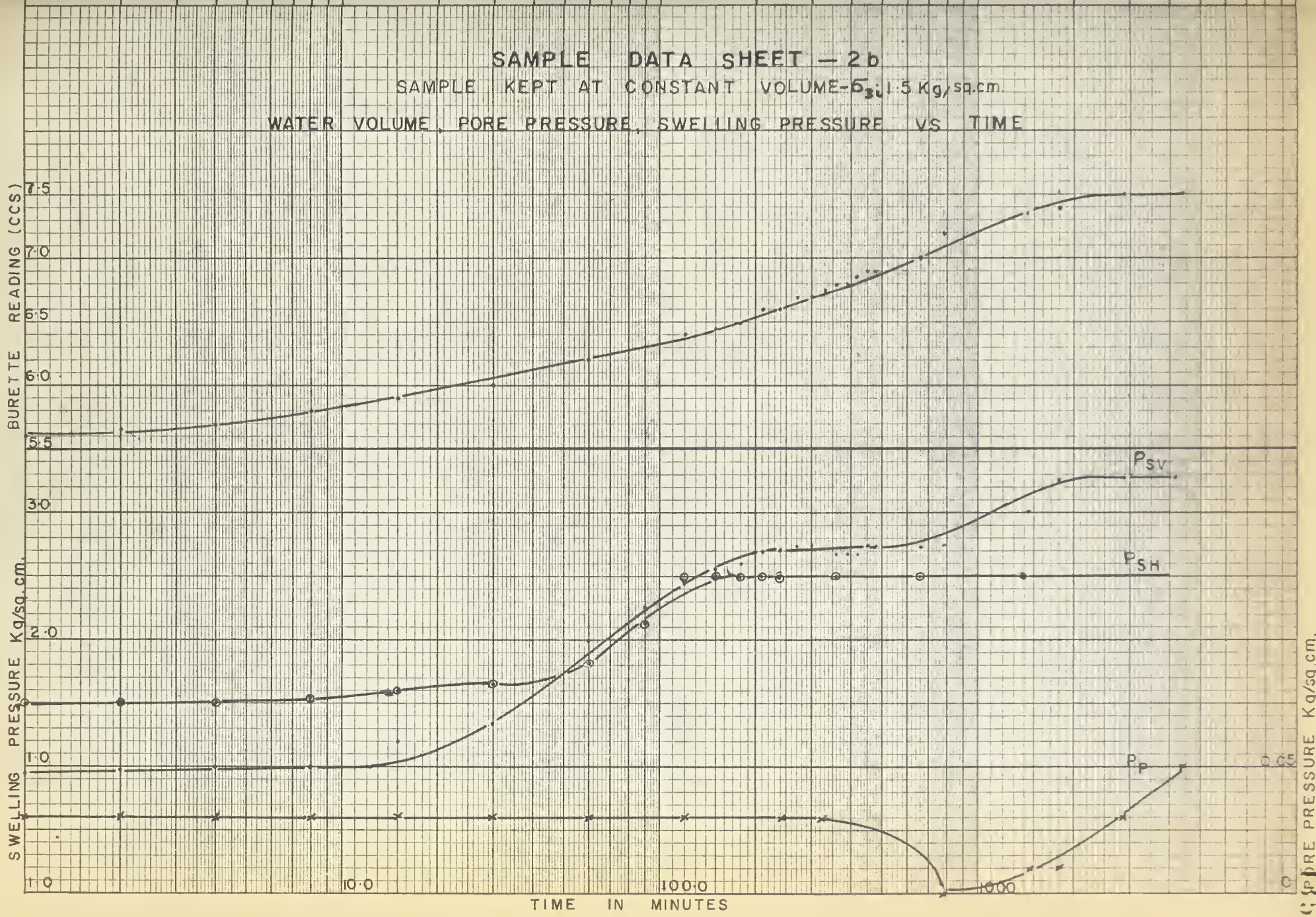
WATER VOLUME VS TIME



TEST. CONSTANT VOLUME TEST

SAMPLE. REMOULDED MORRIN CLAY

DATE. JANUARY 20, 1962


SAMPLE DATA SHEET — 2b
SAMPLE KEPT AT CONSTANT VOLUME-σ_{3i} 1.5 Kg/sq.cm.
WATER VOLUME, PORE PRESSURE, SWELLING PRESSURE VS TIME
BURETTE READING (CCS)
7.5
7.0
6.5
6.0
5.5
SWELLING PRESSURE Kg/sq.cm.
3.0
2.0
1.0
P_{SV}
P_{SH}
P_P
0.05
1.0
10.0
100.0
1000
0
TIME IN MINUTES
PORE PRESSURE Kg/sq.cm.

SAMPLE DATA SHEET - 3

Triaxial Compression

Weight of Piston	2.8 Kg.	TEST	Constant Volume Test
Rate of Strain	1% per hour.	SAMPLE	Remoulded Morrin Clay
Proving Ring Factor-	1 Division = .101 Kg.	DATE	January 25, 1962

Elapsed Time (Min.)	Axial Strain - %	Strain Dial Reading	Corrected Area (sq.cm.)	Stress Dial Reading	Total Load (Kg.)	σ_1 Total	Pore Pressure (Kg/sq.cm.)	Effective Stress σ_1'	Effective Stress σ_3'	σ_1'/σ_3'	ϕ^o	$\bar{A}$
0	0	100	8.04	274	30.44	3.79	0.02	3.77	2.48	1.52	11.9	-
23	0.05	101.4	8.04	342	37.34	4.64	0.11	4.53	2.39	1.89	17.9	.042
28	0.10	102.8	8.05	354	38.65	4.80	0.13	4.67	2.37	1.97	19.1	.048
33	0.15	104.2	8.05	366	39.71	4.93	0.15	4.78	2.35	2.03	19.9	.053
37	0.20	105.6	8.06	376	40.78	5.06	0.16	4.90	2.34	2.09	20.7	.054
42	0.25	107	8.06	383	41.48	5.15	0.18	4.97	2.32	2.14	21.3	.060
62	0.50	114	8.08	410	44.21	5.47	0.25	5.22	2.25	2.32	23.5	.077
78	0.75	121	8.10	429	46.13	5.70	0.28	5.42	2.22	2.44	24.6	.081
93	1.00	128	8.12	439	47.14	5.80	0.32	5.48	2.18	2.51	25.3	.091
108	1.25	135	8.14	445	47.75	5.86	0.30	5.56	2.20	2.53	25.7	.083
123	1.50	142	8.16	450	48.25	5.91	0.26	5.65	2.24	2.52	25.4	.070
137	1.75	150	8.18	455	48.76	5.96	0.23	5.73	2.27	2.52	25.4	.061
153	2.00	156	8.20	458.5	49.11	5.99	0.20	5.79	2.30	2.52	25.4	.052
168	2.25	163	8.23	462	49.46	6.01	0.18	5.83	2.32	2.51	25.5	.046
198	2.75	177	8.28	465	49.80	6.01	0.16	5.85	2.34	2.50	25.4	.040

SAMPLE DATA SHEET

SAMPLE DATA SHEET - 4

TEST	Constant Volume Test
SAMPLE	Remoulded Morrin Clay
DATE	January 25, 1962

Conditions at failure

Volume by Mercury Immersion

Weight Mercury and Tare	1080 gm.
Weight Tare	232 gm.
Weight Mercury	848 gm.
Volume of Mercury	848/13.54 = 62.6 ccs.
Volume of Sample	

Failure Criterion (σ_1'/σ_3) Max.		Axial Strain	1.25%
Total Stress	5.86 Kg/sq.cm.	Pore Pressure	0.30 Kg/sq.cm.
σ_1'	5.56 Kg/sq.cm.	σ_3 Total	2.50 Kg/sq.cm.
σ_3'	2.20 Kg/sq.cm.	ϕ'	25.7°

	Before Compression	After Compression
Weight of Tare - Soil - Water	123.720	107.707
Weight Tare - Dry Soil		77.247
Weight Water	47.27	30.46
Weight Tare	2.083	2.881
Weight Dry Soil	74.37	74.37
Moisture Content	63.6	41.0
Void Ratio	1.87	1.21
Degree of Saturation	97.6	96.8

Computing c and ϕ by the Method of Least Squares

Standard Tests

No.	S_1	S_3	S_1+S_3	S_1S_3	S_1^2	S_3^2
1	1.30	0.09	1.39	.12	1.69	.01
2	1.83	0.30	2.13	.55	3.35	.09
3	2.51	0.80	3.31	2.01	6.30	.64
4	3.82	1.62	5.44	6.19	14.59	2.62
5	6.22	3.21	9.43	19.97	38.69	10.30
Σ	15.68	6.02	21.70	28.84	64.62	13.66

n = 5 S_1 Effective Axial Stress at failure. Kg/cm^2
S_3 Effective All-round stress at failure Kg/cm^2

$$A^2 = \frac{\Sigma S_1^2}{S_3^2} = \frac{\Sigma S_1^2 - (\Sigma S_1)^2/n}{S_3^2 (\Sigma S_3)^2/n} \qquad \frac{64.62 - 49.19}{13.66 - 7.25}$$

$$A^2 = \frac{15.43}{6.41} = 2.41$$

$$A = 1.55$$
$$\sqrt{A} = 1.25$$

$$\text{Sin } \phi = \frac{A - 1}{A - 1}$$

$$= \frac{.55}{2.55} = .2157$$

$$\phi = 12.4^o$$

$$C = \frac{\Sigma S_1 - A\Sigma S_3}{2n\sqrt{A}} = \frac{15.68 - (1.55 \times 6.02)}{10 \times 1.25} = \frac{6.35}{12.5}$$

$$= 0.51$$

c = 0.51 Kg/sq.cm.
ϕ = 12.40°

Computing c and ϕ by the Method of Least Squares

Constant Volume Tests
(Ps included)

No.	S_1	S_3	S_1+S_3	S_1S_3	S_1^2	S_3^2
1	4.81	2.10	6.91	10.10	23.49	4.41
2	4.79	2.09	6.88	10.01	22.94	4.37
3	4.99	2.20	7.19	10.98	24.90	4.84
4	5.90	2.66	8.56	15.69	34.81	7.08
5	6.55	2.98	9.53	19.52	42.90	8.88
Σ	27.04	12.03	39.07	66.30	149.04	29.58

$n = 5$ S_1 Effective Axial Stress at failure Kg/cm^2
S_3 Effective All-round stress at failure Kg/cm^2

$$A^2 = \frac{149.04 - 146.23}{29.58 - 28.94} = \frac{2.81}{0.64} = 4.39$$

$$A = 2.09$$

$$\sqrt{A} = 1.45$$

$$\text{Sin } \phi = \frac{1.09}{3.09} \quad \phi = 20.6^\circ$$

$$= .35275$$

$$C = \frac{27.04 - (2.09 \times 12.03)}{10 \times 1.45} = \frac{1.90}{14.5}$$

$$= 0.13$$

$c = 0.15$ Kg/sq.cm.
$\phi = 20.6^\circ$

Computing c and ϕ by the Method of Least Squares

Constant Volume Tests
(Ps excluded)

No.	S_1	S_3	S_1+S_3	S_1S_3	S_1^2	S_3^2
1	1.38	-0.30	1.08	-.41	1.90	.09
2	1.48	-0.31	1.17	-.46	2.19	.10
3	1.53	-0.30	1.23	-.46	2.34	.09
4	3.15	0.58	3.73	1.83	9.92	.34
5	4.30	1.35	5.65	5.81	18.49	1.82
6	6.05	2.00	8.05	12.10	36.60	4.00
Σ	17.89	3.02	20.91	18.41	71.44	6.44

n = 6 S_1 Effective Axial Stress at failure Kg/cm^2
S_3 Effective All-round stress at failure Kg/cm^2

$$A^2 = \frac{71.44 - 53.37}{6.44 - 1.52} = \frac{18.07}{4.92}$$

$$= 3.67$$

$$A = 1.91$$

$$\sqrt{A} = 1.39$$

$$\text{Sin } \phi = \frac{.91}{2.91}$$

$$.3127$$

$$\phi = 18.20^\circ$$

$$c = \frac{(17.89) - (1.91 \times 3.02)}{12 \times 1.38} = \frac{12.12}{16.56} = 0.73$$

c = 0.73 Kg/sq.cm.
ϕ = 18.2°

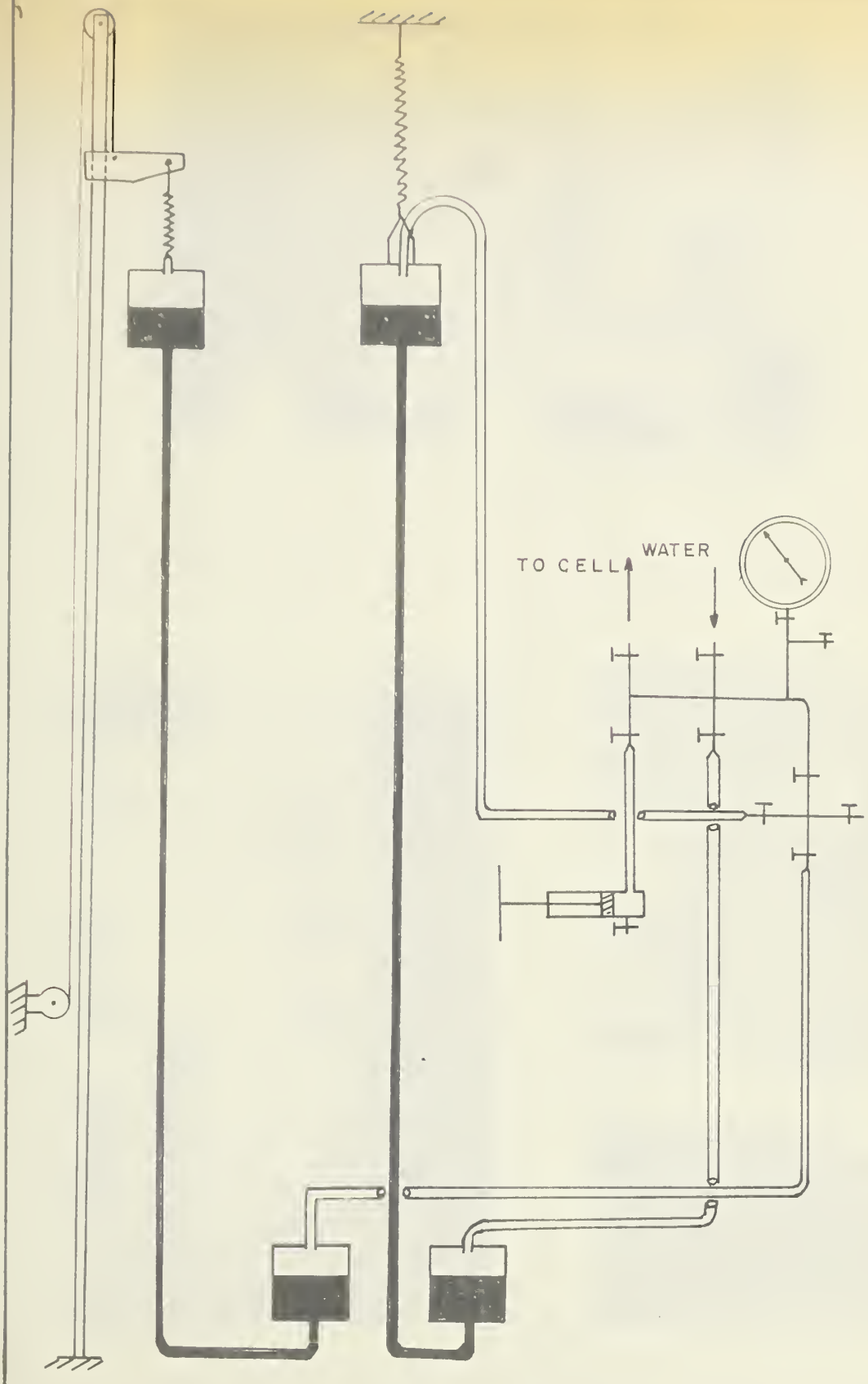
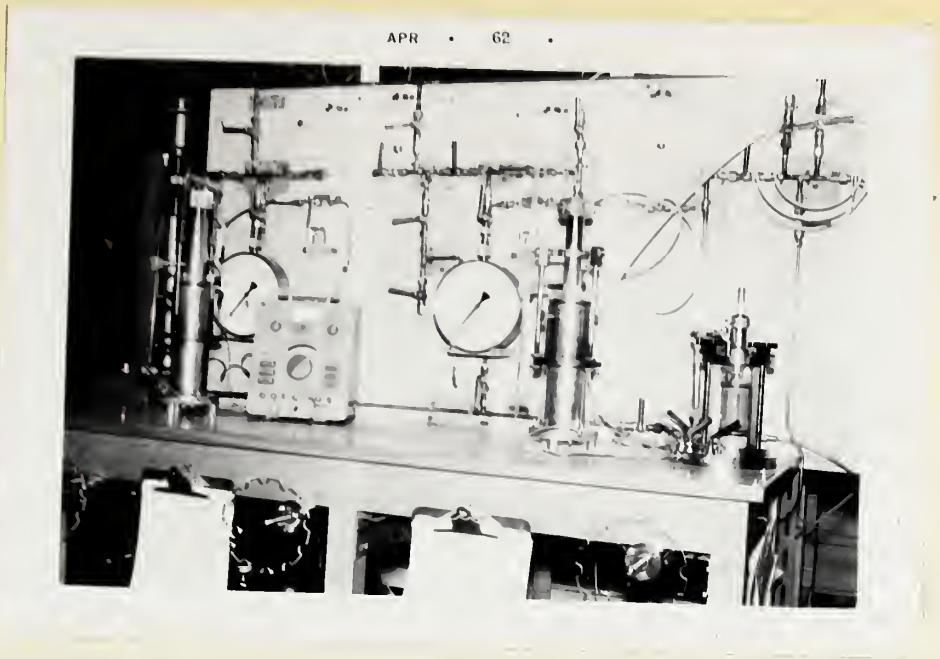


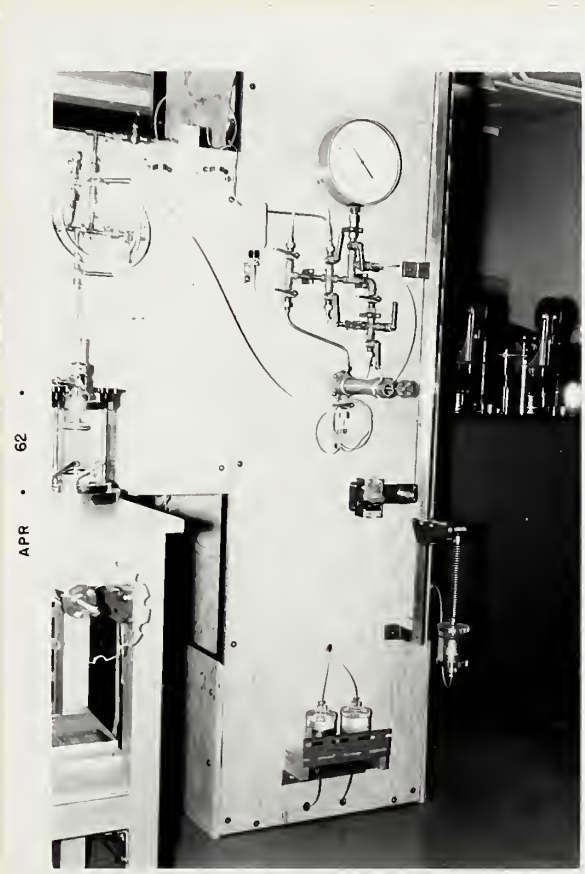
FIG. 23

THE SELF-COMPENSATING MERCURY CONTROL

AUGUST 20, 1961



(a) GENERAL VIEW OF THE CONTROL PANEL AND CELLS



(b) THE CONSTANT PRESSURE MERCURY CONTROL

(c) A SAMPLE CONSOLIDATING IN THE STANDARD CELL

PLATE VI

APPENDIX IV

NOTES ON SHEAR STRENGTH AND SHEAR STRENGTH TESTING OF COHESIVE SOILS

The Principle of Effective Stress.

a. Application to Saturated Soils. The principle of effective stress was first implicitly enunciated by Terzaghi in 1925 as a result of his work on the theory of consolidation and on the shear strength of cohesive soils. In his work leading up to the discovery of his important principle, Terzaghi assumed a purely cohesive material, fully saturated with a single pore fluid, and this assumption must always be borne in mind whenever an attempt is made to apply the principle to any material.

The principle of effective stress is expressed as follows by Bishop (1960).

> "(a) The change in volume of an element of soil depends not on the change in total normal stress applied, but on the difference between the change in total normal stress and the change in pore pressure.
> (b) The maximum resistance to shear on any plane in the soil is a function, not of the total normal stress acting on the plane, but of the difference between the total normal stress and the pore pressure."

In each of the above cases the effective normal stress is given by Terzaghi's equation:-

$$\sigma' = \sigma - u \qquad \text{----(4)}$$

The principle of effective stress and Terzaghi's equation have been subjected to rigorous examination by several investigators, and their findings show that the assumption on which this principle is based cannot be overlooked. Lambe and Whitman (1959) point out that the behaviour of a soil is not necessarily governed by the average stress acting at the points of contact between the soil grains, since the expression for the shear that governs the resistance to rolling or sliding is $\bar{\bar{\sigma}}\, a_c$, while the expression for the stress which causes rearrangment of the soil particles is $(\sigma - u)$. Thus if effective stress is defined as the stress which controls soil behaviour, the appropriate equation to be used will depend upon the property of the soil which is under investigation. Table XI lists the equations that are applicable when certain properties of the soil are being investigated.

TABLE XI

EQUATIONS THAT MAY BE USED TO EXPRESS EFFECTIVE STRESS

Soil Property under Investigation	Appropriate equation
Compression under moderate pressure	$\sigma' = \sigma - u$ (4)
Shear Resistance	$\sigma' = \sigma - u(1+a_c)$ (5)
Compression under high pressure	$\sigma' = \frac{1}{a_c}(\sigma - u)$ (6)

Equation (4) above is the most meaningful when the compression behaviour of soils is studied at moderate pressure, and is therefore appropriate for most of the problems with which the soils engineer is faced.

When the shearing resistance of a cohesive soil is the property under study, the contact area between the mineral grains (a_c) is assumed to be negligible, and equation (5) reverts to the form of equation (4). This assumption of negligible contact area has been proven to be valid by Skempton (1960), who has shown that equation (5) is not generally adequate for fully expressing the effective stress acting in saturated materials, but at pressures normally encountered in engineering problems, the value of a_c is less than 1%, and so for fully saturated soil at these pressures, Terzaghi's equation (4) is quite valid. If the material is incompressible and purely cohesive, then Terzaghi's equation is rigorously true.

b. Application to Expansive Soils. Terzaghi's equation was derived from studies on granular soils and clays of low plasticity. Lambe (1959) presents an argument to show that for these soils, the expression for effective stress given as equation (5) is meaningless because of (1) the unknown contact area between the soil grains, (2) possible adhesion between the soil particles, (3) the uncertainty as to the meaning of pore pressure, (4) the lack of theory relating stress

to soil behaviour.

In equation (5) there are two hidden assumptions which are not wholly justified when applied to cohesive soils. The first is that the only net force transmitted between adjacent soil particles is derived from externally applied loads, and the second is that all forces carried by the soil skeleton, are transferred through the horizontal component of area of contact between mineral particles, and that all pore water pressure is transmitted through the horizontal component of water area going through the zone of contact between the mineral particles.

The first assumption is considered to be generally true in the case of cohesionless soils, but in the case of cohesive soils, very significant forces of electrical attraction or repulsion can be generated between the mineral particles, and this can be the source of a net force on the soil skeleton which may differ greatly from the externally applied force.

In the second case one error arises from the fact that in clay soils, the individual mineral particles may be entirely separated by a water film, in which case the inter-particle forces are transmitted by electrical forces which act through the water, and so the horizontal component of contact area must be extended to include the entire horizontal area over which the influence of the electrical forces is

felt. In some cases there may be mineral to mineral contact between the soil particles, and here the contact area can transmit stresses, but the extent of this contact is uncertain.

Another error arising in the second case is the uncertainty as to the meaning of pore pressure. In cohesionless soils, all of the water in the voids is essentially of the same pressure, and no uncertainty arises, nor is there any difficulty in obtaining the magnitude of this pressure by means of a piezometer installed in the soil mass. In clays however, the water in the double layer is acted upon by strong electrical forces originating in the mineral particles of the soil; the force of electrical attraction drops rapidly as distance from the particle increases, causing a variation of the pore pressure, which will depend upon the point at which the measurement is made. For expansive soils pore pressure may be presumed to mean the pressure in the pores, measured at a point where the electrical forces are least effective, but such a point is non existant.

Lambe and Whitman (1959) have proposed that for expansive soils equation (5) (Table XI) should be modified to read

$$\sigma = \bar{\bar{\sigma}} a_s + u (1 - a_u) \qquad ----(7)$$

and that the value of u used may be the value obtained in a piezometer, provided that the fluid in the instrument has as nearly as possible the same electrolyte composition and ion concentration as

the pore water*, and providing that sufficient time is allowed for the establishment of an ion diffusion equilibrium.

The variation in water pressure in the double layer is believed to be uniquely related to and accounted for by the interparticle electrical forces, and so the uncertainty of the value of u is compensated for by use of the proper definition of $\bar{\bar{\sigma}}$. This of course is just as difficult to determine as the true value of u .

From the above it would appear that for swelling soils, there are uncertainties of appreciable magnitude which are present in any attempt to estimate their shear strength or compressibility. In spite of this, the principle of effective stress is the best tool available for predicting the shear strength and volume change characteristics of these soils. Lambe and Whitman (1959) feel that the trouble lies in the fact that the behaviour of swelling soils is being forced to fit the concept of effective stress, while the solution may possibly be in an entirely new concept.

c. Application to Unsaturated Soils.** Experimental work has shown that in order for the principle of effective stress to be

* Bishop (1960) points out that no experimental values have been given to show that failure to satisfy this condition leads to a significant error.

** Unsaturated soils are taken as those having a degree of saturation less than 85%.

applied successfully to partially saturated soils, a modification is again needed for the expression given in equation (5). Partially saturated soil is a three or four phase system in which the gas, air and water in the pore spaces of the soil, may be in equilibrium at pressures which may differ considerably because of surface tension.*

Bishop (1960) has tentatively proposed the following expression for effective stress in partially saturated soils

$$\sigma' = \sigma - u_1 + \chi (u_1 - u_2) \qquad ----(8)$$

In this expression χ is a parameter which has a value of unity for saturated soils, and a value of zero for dry soils. In both of these extreme cases equation (8) reverts to the form given in equation (4) (Table XI). Intermediate values which this parameter will obtain depend mainly on the degree of saturation of the soil, but will also be influenced by such factors as soil structure, cycles of wetting and drying, and changes in stress leading to a particular value of saturation. Values of this parameter may differ depending upon whether shear strength or volume change properties are being investigated. Fig. 24 gives what Bishop believes to be the probably general form of the relationship between the parameter χ and the degree of saturation of the soil.

*Pore water pressure is always less than pore air pressure.

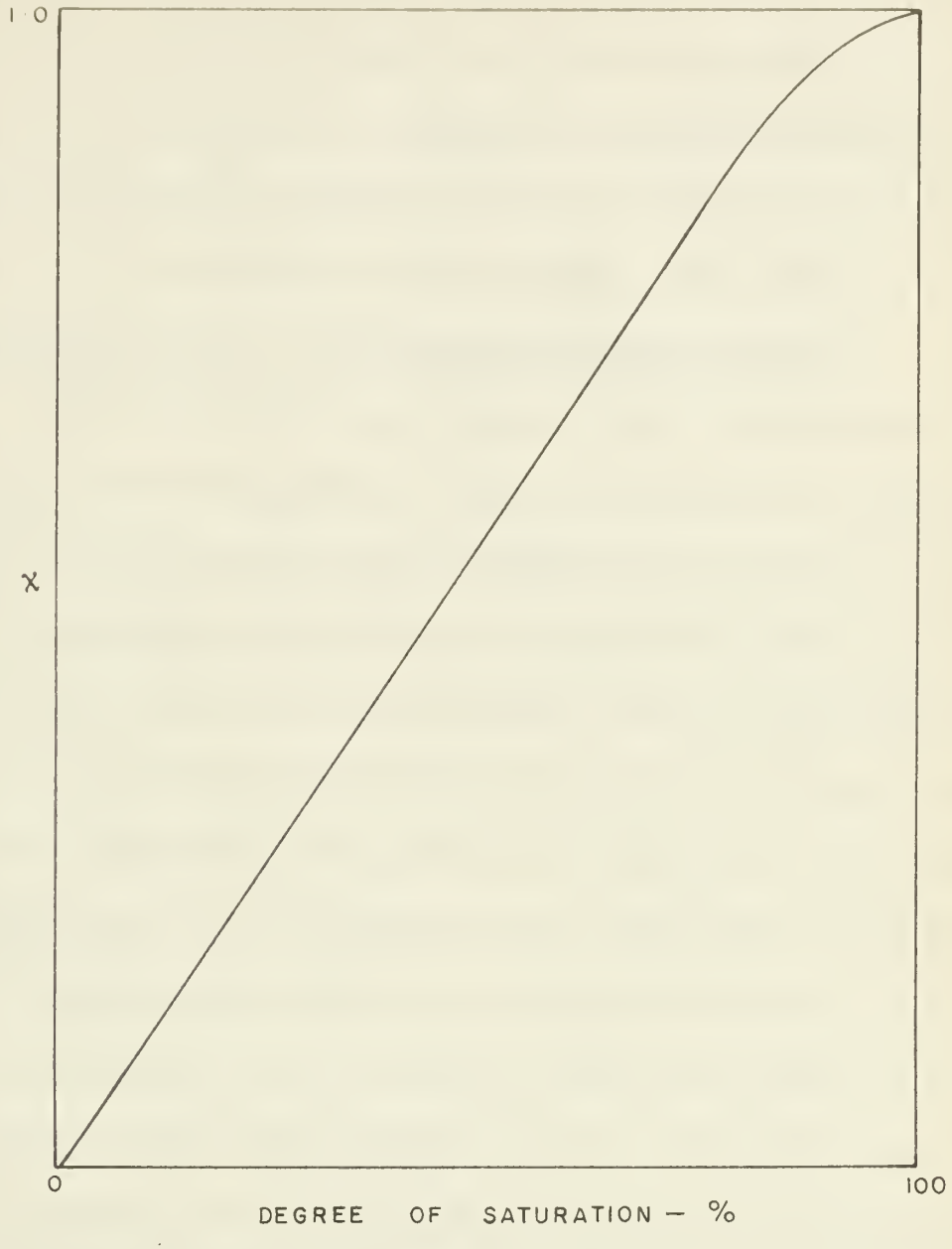


FIG. 24

PROBABLE GENERAL FORM OF THE RELATIONSHIP BETWEEN THE PARAMETER X AND DEGREE OF SATURATION

(BISHOP 1960)

Equation (8) has been experimentally confirmed, and involves the measurement of pore water and pore air pressure. It serves as proof that the principle of effective stress as expressed by Terzaghi's equation (4) is unsuitable for application to partially saturated soils, and must be suitably modified to fit those conditions which do not agree with the assumption on which the principle is based.

Pore Water Pressure -- Its Measurement and Importance

The measurement of pore pressure in routine laboratory strength tests is comparatively recent, and in most of the equipment used, it involves no flow of the pore fluid to or from the pore spaces. The disadvantages of flow of the pore fluid are the considerable time lag involved in soils of low permeability, and the considerable variation in pore pressure which would be encountered in ordinary samples. Several methods of measuring pore pressure have been used with varying success, the principal difference between them being the type of null indicator used, and the type and position of the porous disc. The water-mercury system seems to be the most flexible and reliable for normal or slow rates of testing, and involves the balancing of the pressure of the pore water against the null indicator, to show that the condition of no flow is satisfied.

Measurement of pore pressure from the base of the sample has been generally adopted because of its relative simplicity and con-

venience in studying the build up and dissipation of pore pressure, when stresses are applied to the soil sample. Base measurement is very convenient and practical in tests on silts and sands, and other soils of relatively low permeability, but in clays, appreciable pore pressure gradients* may exist, and persist for long periods, depending on the permeability, the degree of saturation, and the specimen size.

To overcome these drawbacks, the tests are run at rates slow enough to allow equalization of the pore pressure over the sample. Vertical filter paper strips placed around the sample, and three or five wool wicks inserted in the sample, are used to accelerate the equalization process.

Tests have been made using diagonal probes inserted in the zone in which failure is considered most likely to occur, but these are technically difficult to operate, and are only advantageous if the rate of testing necessitates quick measurement of pore pressure.

The success of pore pressure measurement depends upon the extent to which the measuring system is de-aired. Thorough de-

* Pore pressure gradients may be caused by initial non uniformity of pore pressure, (as in a sample compacted in layers), non uniformity in stress during shear due to end restraint, and non uniformity in strain due to a tendency for failure to occur in a narrow zone.

airing is a difficult but essential task, and much time and patience may be needed to achieve the desired end. When consolidated-undrained triaxial tests with pore pressure measurements are to be made on saturated samples, dry free air which is trapped between the sample and the membrane can be expelled by flushing out this space with de-aired water. Since no water will be transferred from the saturated porous disc to the sample, the pore size of the disc is controlled only by its permeability and its resistance to entry by the fines of the soil, and it may be made from metal gauze, sintered bronze or ceramic material.

The measurement of pore pressure in undrained tests on partially saturated samples is not as simple. Any sample of soil, (undisturbed, remoulded or compacted), which is capable of standing unsupported, has negative pore pressures of varying magnitudes depending on the type, stress history and moisture content of the soil. When such a sample comes in contact with a saturated porous disc, it will immediately begin to take up water, unless the negative pressure can be balanced by the pressure measuring system. In order for this to take place, the negative pore pressure in the soil must not exceed the air entry value of the porous stone, and the negative pressure in the measuring system must not go below minus thirteen pounds per square inch (At sea level). If the first

condition is not satisfied, air will enter the porous stone as water is drawn into the sample, and the efficiency of the measuring system will be reduced. Failure to meet the second requirement will result in cavitation in the measuring system, and water will be drawn from the system through the porous stone into the sample. Both conditions will cause a time lag in the measurement of the pore pressure, due to the wetting of the sample near the porous disc.

For partially saturated samples, a third condition must be met. These samples contain both water and air in the voids of the soil, and the difference between the pore air pressure and the pore water pressure should not exceed the air entry value of the porous disc.

The use of High Air Entry Ceramic Discs for Measuring Pore Pressure.

The measurement of pore pressure in the triaxial test employing the use of porous discs having a high air entry value is relatively new. This technique has made possible the easy measurement of negative pore pressure, has helped in the verification of some proposed theories, and has greatly aided in reducing the discrepancies and errors that frequently arise in routine testing.

The air entry value of a porous material, fully saturated by a wetting phase, is that pressure at which a non wetting phase in con tact with the material, starts to displace the wetting phase, thus

initiating dissaturation.

If it is desired to calculate the air entry value of a disc of a given material, the average diameter of the openings in the disc must be known, as well as the surface tension of the saturating fluid. If the openings in the disc are considered to be capillaries of radius R and the surface tension of the saturating fluid is assumed to be T dynes per centimeter of length, then assuming a zero angle of contact between the fluid and the walls of the openings, the pressure required to empty the capillaries would be:-

$$p = \frac{2T}{R} \text{ dynes/sq.cm.}$$

At the Imperial College of Science and Technology -- London* two porous ceramics have proved satisfactory from the point of view of strength and air entry values, and have been used in several investigations. Table XII lists the relevant physical properties of these ceramics.

TABLE XII

Physical Properties of Two Ceramics

Type of Ceramic	Aerox Celloton Grade VI	Doulton Grade P6A
Porosity	46%	23%
Air Entry Value	30 lb/sq.in.	22 lb/sq.in.
Permeability	2.9×10^{-6} cm/sec	2.1×10^{-7} cm/sec

* Alpan (1959) Bishop (1960)

The discs used are usually three-eighths of an inch thick and are fitted into a recess in the pedestal of the standard cell, and sealed with epoxy resin, "Twin Bond", in order to prevent the possibility of air leaking between the ceramic disc and the pedestal. To de-air this unit, it may be saturated while under a vacuum, or the cell may be assembled and filled with de-aired water at a pressure of about 120 psi, and left for several hours to allow all air present to be dissolved. At the end of this period, the measuring device is opened and all the water is flushed through.

The efficiency of the de-airing procedure and the freedom of the system from leaks can be checked by exposing the disc to the open air, wiping it free from water, and applying to it a negative pressure of up to minus 13 psi by means of the screw-control. The movement of the water mercury interface of the null indicator will reveal whether the system is leak-free and de-aired.

The Measurement of Pore Air Pressure

Whenever the pore pressure of a partially saturated soil is to be measured, it becomes necessary to measure the pore air pressure in addition to the pore water pressure, so that the effective stress may be evaluated from Bishop's equation (8). Measurement of pore air pressure is made by means of a porous element, which provides free communications between the air voids of the soil, and

the pressure measuring system. This porous element should have a lower attraction for water than for air, and its volume, together with that of its air filled connections to the null indicator, should be small, since the high compressibility of air would demand a compensating adjustment to the null indicator if the air volume is too large.

Satisfactory results in the measurement of pore air pressure have been obtained at the Imperial College of Science and Technology -- London (Bishop 1960) by the use of a single layer of glass fibre cloth woven from coarse dressed yarn, made in the form of a disc and placed on top of the sample. From this disc a polythene tube connects to the null indicator. When it is required to measure both air and water pressure from the same end of the sample (as in the case of a disipation test) a length of glass fibre tape is wrapped around the sample just above the base, and a connection is made through the rubber membrane to the null indicator. This arrangement works well when the soil is at optimum moisture content or dryer, but at higher degrees of saturation the air in the soil appears to be discontinuous, the difference between pore air and pore water pressure is small, and water is likely to enter the air measuring system.

B29798